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NPDES COMPLIANCE FLOW MEASUREMENT MANUAL

U.S. Environmental Protection Agency

September, 1981

by:

David L. Guthrie, P.E.

Office of Water Enforcement and Permits
Enforcement Division
Compliance Branch

DISCLAIMER

This manual has been reviewed by the Office of Water Enforcement and Permits, U.S. Environmental Protection Agency, and approved for publication. Mention of trade names or commercial products constitutes neither endorsement nor recommendation for use.

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The author also would like to acknowledge the efforts of the engineering staff of JRB Associates, Inc. for their technical assistance in preparing some of the background materials used herein.

NPDES COMPLIANCE FLOW MEASUREMENT MANUAL

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FOREWORD

The subject of flow measurement is not new. One of the first to recognize the relationships between area, velocity, and quantity of flow was Hero of Alexandria about 150 B.C.! The science of hydraulics evolved during the Renaissance -- Torricelli (1608-1648) developed the concept of velocity head in the early 1600s.[1] In fact, many of the formulas presented in this manual have their roots in the 18th and 19th centuries. Several scientists have made outstanding contributions to the field of hydraulic engineering in this period.

Flow measurement is now an integral part of the National Pollutant Discharge Elimination System (NPDES).[2] To comply with the permit requirements established under NPDES, the wastewater discharger or permittee must determine the quantity of wastewater generated.

Most effluent permit requirements are expressed in terms of mass loadings instead of pollutant concentrations to enable the regulatory agency to determine more accurately the permittee's compliance status. Mass loadings are quantities of pollutants per some unit time. For example, kilograms per day is a common mass loading expression.

Pollutants vary from the standard indicators of aquatic environmental quality -- pH, temperature, color, odor, suspended solids (SS), biochemical oxygen demand (BOD), chemical oxygen demand (COD), turbidity, etc. -- to some of the more toxic substances -- pesticides such as dieldrin, DDT, and mirex, heavy metals such as mercury, cadmium and selenium, and carcinogens such as PCBs and asbestos. The pollutants and limitations specified in NPDES permits vary from permit to permit, but in most cases they are not expressed in concentrations except where the flows are extremely variable and usually small.

The Compliance Branch, EPA Office of Water Enforcement and Permits, has compiled several compliance inspection manuals. They are:

- NPDES Compliance Sampling Inspection Manual, 1979.
- Interim NPDES Compliance Biomonitoring Inspection Manual, October 1979.
- NPDES Compliance Evaluation Inspection Manual, January 1981.

These manuals present the legal considerations and ramifications of compliance inspections, planning inspections, inspection types, general information on health and safety, and post inspection activities including completion of inspection forms. These procedures and discussions will not be repeated in this Flow Measurement Manual. For more information, please refer to any of the above manuals.

The NPDES Compliance Sampling Inspection Manual includes a small section on flow measurement, but its scope is limited. This Flow Measurement Manual is designed to furnish enough additional information on flow hydraulics to whet the appetite of the curious investigator and lead him/her in quest of more detail elsewhere.[24]

It is anticipated that field surveillance personnel from State and Federal environmental regulatory agencies will represent those interested in flowrate measurements for compliance monitoring purposes. This manual is directed towards meeting the needs of this group of students. To the extent possible, material is presented with emphasis on practical applications. Theory is presented only for the purpose of conveying basic concepts; technical references are cited for those students interested in pursuing the more theoretical considerations and exploring the field at length.

INTRODUCTION

Flow data are needed for two basic reasons. First, as previously explained, mass loadings are usually specified in NPDES permits. Since the flow multiplied by the unit concentration (and by the appropriate conversion factor) is equal to the mass pollutant loading, properly obtaining flow data is as critical to calculating pollutant loadings as good sampling and analysis techniques are to ascertaining pollutant concentrations.

Second, the current compliance strategy depends heavily on the submittal of self-monitoring data by each permit holder. The NPDES inspection, then, should verify the data collected by the permittee, support any enforcement action if necessary, and eventually provide a basis for NPDES permit reissuance or revision.

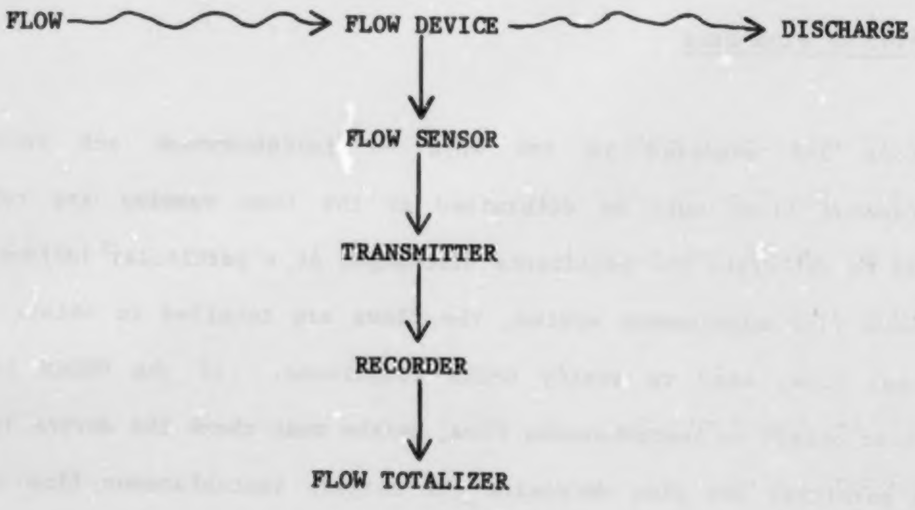
Collection of Flow Data

Flows are measured in two ways — instantaneous and continuous. Instantaneous flows must be determined at the time samples are taken for analysis to calculate the pollutants discharged at a particular instant. In a continuous flow measurement system, the flows are totalled to obtain a value for total flow, used to verify NPDES compliance. If the NPDES inspector desires to obtain an instantaneous flow, he/she must check the device installed by the permittee and also determine the correct instantaneous flow by using his/her own measurement technique and/or portable equipment. Such equipment will be discussed later. If the NPDES inspector desires to obtain a value for total flow, he/she must use a continuous flow measurement system.

A word of caution should be inserted at this point. It is very common for authors to use the term "flowrate" when describing flow. To avoid confusion, the terms flow and velocity will be used in this text. Flow is the quantity of fluid per unit time (e.g., gallons per day), whereas velocity is a distance over unit time (e.g., feet per second).

Flow Measurement Systems

A flow measurement system is required to obtain continuous flow data. It is constructed so that it has predictable and measureable hydraulic responses related to the flow of the water or wastewater through it.[3] A standard flow measurement system consists of the following five components:



The primary element in the system is, of course, the flow device by which the flow can be measured. Flow devices include weirs, flumes, venturi or orifice meters, magnetic flow meters, among others.

The flow sensor measures the height (or head) of the water or wastewater over or in the flow device. The flow sensor can be a probe, a float, or a series of pressure cells, for example.

The flow sensor is connected to the flow recorder by means of a transmitter. The transmitter can include electrical conductors, wires or cables, etc.

Now that the signal has arrived at the recorder, it is transformed into a unit of flow by a mechanical, electromechanical, or electronic system. This system displays the flow per unit time once the flow measurement system has been initially calibrated. Common displays are strip charts, circular charts, or inputs into a mini-computerized data file by data elements.

More advanced systems include a flow totalizer attached to the recorder. Basically, the flow totalizer displays total flow on a real time basis, calculated by taking the integral of the value of the height of water over the gage over time.

It is important that the NPDES inspector check the permittee's flow measurement system. Is instantaneous or continuous flow measurement required in the NPDES permit? If the flow must be measured by a flow measurement system and there is none on-site, then the NPDES inspector should install a temporary device. This task is difficult and time-consuming and should be avoided if possible. If a primary flow device has been installed by the permittee, and if only a reading of instantaneous flow is desired, then only a flow sensor, transmitter and recorder should be installed. Of course what is required depends on the conditions stated in the NPDES permit.

A brief discussion should be inserted here about precision and accuracy of flow measurement systems. Precision and accuracy have distinctly different meanings. Precision is used to describe data reproducibility, or the ability to obtain approximately the same data from repeated measurements of the same quantity. Accuracy is the lack of deviation from a known true value. For example, if a series of measured flow values are 3.0, 3.1, 2.95, 2.98, and 3.05 million gallons per day (mgd), but the true value is 3.75 mgd, the measurements are very precise but not accurate. If the measured values are 3.6, 3.8, 3.75, 3.7, 3.85 mgd, they are fairly accurate but not precise.

The accuracy of flow measurement devices varies widely with the device, its location, the environmental conditions (e.g., is it submerged?) and other factors such as maintenance and proper calibration. Normally error greater than ±10% is considered unacceptable for NPDES compliance assurance purposes.[3] However, error values of ±25% are not uncommon in some cases. In fact, one EPA region determined that approximately 50 percent of those sampling stations surveyed had no flow measurement devices of any kind, and it was frequently necessary to install temporary flow measurement equipment.[4]

When evaluating a flow measurement system during a compliance inspection, the inspector must consider four items:

(1) Does the system/device measure the entire flow? -- Sometimes the permittee's flow measuring device is too far upsteam.

(2) Is the permittee's system accurate? -- There are two ways to check the permittee's flow measurement system. First, the inspector can take an independent flow measurement with other equipment or by another method. Second, the inspector can check existing equipment to see if it is properly installed, calibrated, and in good working order.

(3) If the permittee's equipment is found to be inaccurate and the problems cannot be corrected, should new equipment be installed? -- This is a recommendation that the inspector should include on the Deficiency Notice* or suggest during the post-inspection debriefing with the permittee.

(4) Does the permittee have a regular calibration and maintenance program for the flow measurement equipment? -- If not, such a program should be recommended.

Methods/Types of Flow Measurement

There are many methods and types of flow measurement and flow measurement devices. This manual will not discuss all of them in detail, but it will present the ones most commonly seen in the field. The most common methods are listed in Table 1. They vary in sophistication from the "throw-the-pingpong-ball-in-the-stream-method" to radioactive tracer studies. The former method is a last resort method to be used when nothing else is available, and the latter method is not recommended because it may have an adverse affect on the environment.

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*The Deficiency Notice (EPA Form 3560-4) is used by the inspector to alert the permittee to deficiencies in their self-monitoring procedures.

Three hydraulic situations will be encountered in the field:

- Open channels
- Closed conduits
- Outfalls

Types of flow measurement applications will now be described to measure flows in each of these situations.

TABLE 1

FLOW MEASUREMENT METHODS

1. Weighing the discharge
2. Volumetric discharge measurement
3. Sump pump calibration
4. Orifices
5. Standard weirs
6. Parshall flumes (Venturi flume)
7. Palmer-Bowlus flumes (critical-depth meters)
8. Nozzles (contracted openings)
9. Pitot tubes
10. Water height measurement and calculation methods
11. Venturi meters
12. Open-pipe methods (California and Purdue)
13. Tracers (chemical and radioactive)
14. Current meters
15. Magnetic flowmeters
16. Acoustic flowmeters
17. Electrical methods
18. Salt-velocity and dilution methods

BASIC METHODS

There are four basic methods for determining flow -- weighing the discharge, volumetric methods, sump pumps, and orifice buckets. They are all useful but rather primitive. However, they are convenient and fast.

Weighing the Discharge

This procedure is easy and quick. First, a container must be selected in which to collect the discharge. Its size depends on the relative volume of discharge. This method should only be used primarily for small flows in the range of 25 gallons per minute (gpm) or less. Another estimating value to determine if this method should be used is the container should take more than two or three seconds to fill. The longer the container takes to fill, generally the more accurate this method will be. A handy container is a standard plastic or metal bucket.

After the container has been selected, weigh it empty. This weight is called the TARE WEIGHT. After weighing, collect a quantity of water or wastewater while using a stopwatch to time how long the container takes to fill. Then the weight of the full bucket minus the tare weight equals the weight of the fluid. To convert the fluid's weight to gpd, use the factor 8.347 lb/gal water.

Example: If a bucket takes 5.1 seconds to fill, the tare weight is 1.4 lbs., and the total weight is 16 lbs., what is the flow?

$$16 - 1.4 = 14.6 \text{ lbs. water} \times \frac{1}{8.347 \text{ lbs/gal}} = 1.75 \text{ gal}$$

$$\frac{1.75 \text{ gal}}{5.1 \text{ sec}} = 0.343 \text{ gal/sec} \times 60 \text{ sec/min} = 20.6 \text{ gpm}$$

Volumetric Methods

Volumetric flow measurement methods have the advantage of being easy. A volumetric measurement is simply a measurement of the amount of time it takes to fill a container. Different types of containers can be used depending on their availability.

Some of them are listed below with their corresponding volumetric formulas.

TABLE 2

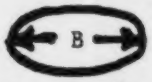
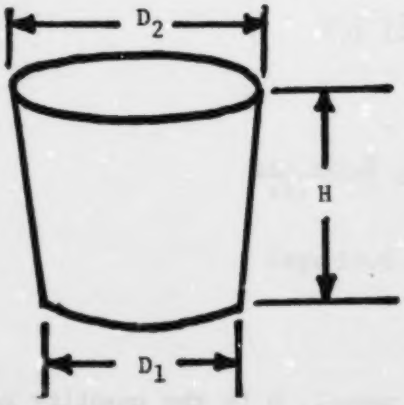
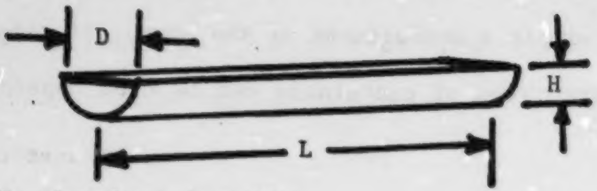
Container Type	Formula
Sphere	$V = 1/6\ \pi\ D^3$
Right Cylinder	$V = 1/4\ \pi\ D^2 h$
Rectangular Cylinder	$V = HLW$
Triangular Cylinder	$V = 1/2\ HBL$
Elliptical Container	$V = \pi\ BDH$ (where B is )
Frustrum of a Cone 	$V = \frac{\pi}{12} H(D_1{}^2 + D_1D_2 + D_2{}^2)$
Cone	$V = (1/12)\ \pi\ D^2 H$

TABLE 2 (Continued)

Container Type	Formula
Parabolic Container	V= (2/3) HDL



In the above formulas,

D = diameter
h, H = height
L = length
W = width
B = base

For this method to approach accuracy, the minimum filling time should be about 10 seconds, and the flow should be less than 30 gpm. Fill the container three times while timing it with a stopwatch, and average the results.

Example: It takes 15.2, 15, and 15.8 seconds to fill a bucket with the dimensions, D_1 = 8 in., D_2 = 10 in., H = 11 in. What is the flow?

$$V = \frac{\pi}{12} H(D_1^2 + D_1D_2 + D_2^2) = \frac{3.1416}{12} (11)\ (8^2 + (8)(10) + 10^2)$$

$$V = 703 \text{ in}^3 \times (1 \text{ ft}^3/1728 \text{ in}^3) = 0.41 \text{ ft}^3$$

$$t = \frac{15.2 + 15 + 15.8}{3} = 15.33 \text{ sec.}$$

$$Q = 0.41 \text{ ft}^3/15.33 \text{ sec} = 0.027 \text{ cfs} \times \frac{0.646 \text{ mgd}}{1 \text{ cfs}}$$

$$Q = 0.0173 \text{ mgd} = 0.017 \text{ mgd (or even 0.02 mgd)}$$

In this example and in the rest of this manual, Q is the quantity of water or the flow.

Sump Pumps

Often, the NPDES inspector will have to calculate flow based on the discharge or overflow from a sump pump. Many pumping stations still discharge raw or disinfected sewage directly to watercourses, so a method to calculate flow by using the sump pump is very useful. Because many pumping station discharges have either underwater (submerged) or inaccessible outfalls, the best method calculates flow in direct proportion to the kilowatt hours used by the station.

A summary of the sump pump method follows. The entire method, with corrections by the author, appears as appendix I.[5]

If the flow, $Q \propto$ kwh used*, then the volume pumped per pump cycle is:

$$V_p = V_s + Q_i t_p$$

where:

V_p = volume pumped/cycle (gal)

V_s = storage volume of the wet well (gal)

Q_i = flow of the influent sewer (gal/sec)

t_p = time pump runs/cycle (sec)

---

* α means "is directly proportioned to"

The error introduced is the flow into the sump during the pump cycle. To correct this error, a station constant must be calculated, based on the station's electrical meter:

$$C = \frac{1{,}000\ V_s \left(1 + \frac{t_p}{t_f}\right)}{nK_h}$$

where:

C = station constant (gal/kwh)

t_f = time required for wet well to fill (sec)

n = number of revolutions made by the meter disc per one pump cycle

K_h = meter constant (watt-hours/revolution)

Thus the flow can be calculated:

$$Q_{avg} = \frac{C\left(E_{t_1} - E_{t_0}\right)}{t}$$

where:

Q_{avg} = average flow for the period (gpm),

C = station constant (gal/kwh),

E_{t_0} = kwh reading at the beginning of the period,

E_{t_1} = kwh reading at the end of the period, and

t = elapsed time between observations E_{t_0} and E_{t_1} (min).

The meter should be read at an interval greater than one hour.

If the instantaneous flow is desired, it is equal to the flow of the influent sewer. To calculate the Q_i (see Appendix I):

$$Q_i = \frac{V_s}{t_f}$$

where:

Q_i = flow of the influent sewer (gal/sec)
V_s = storage volume (gal)
t_f = time required to fill the sump (sec)

However, three notes of caution should be mentioned:

(1) When both pumps are operating together instead of alternately in stations with more than one ejector pump, the force main will choke, decreasing the head loss (h_L) and thus decreasing the efficiency.

(2) Pay attention to meter multipliers (e.g., one notch = 4 kwh).

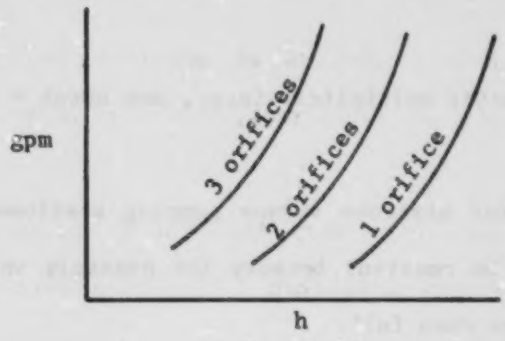
(3) In pneumatic ejector stations versus pumping stations, the "batch" volume of discharge will be constant because the pressure vessel discharges under forced air pressure when full.

Orifice Bucket

The Orifice Bucket technique for measuring wastewater flow is a step up from the "bucket and stopwatch" technique. It involves the use of an orifice bucket, which is basically a five gallon bucket modified by cutting holes in the bottom and plugging the holes with rubber stoppers for calibration. The flow is a function of the depth of wastewater in the bucket, or $Q = f(h)$, where h is the hydraulic head.

A graduated piezometer tube on the outside is used to read the water levels. A dispersion device like a screen should be mounted in the bucket to reduce direct velocity impingement on the orifices.[4] The orifice bucket can be used to calculate a range of flows from 7 to 100 gpm.

A calibration or rating curve family should first be developed in the laboratory by removing one of the rubber stoppers and determining the flow rate through the orifice at different constant heads with a known, variable water source.[6] It should generally look like the following:



A constant head is required to achieve accuracy with the orifice bucket method, so it should not be used for variable flow discharges.

WEIRS

As previously stated, there are three basic types of hydraulic conditions -- closed conduits, open channels, and outfalls. Weirs are a commonly used flow measurement device and are found in many field situations. They are one of the three kinds of devices normally employed to measure open channel flow. The other devices, which will be discussed later, are flumes and meters. Weirs are the most simple and reliable of the three devices.

Sharp-Crested Weirs

There are two basic categories of weirs -- sharp crested and broad-crested. In each of these two categories, there are many types of weirs.

A sharp-crested weir is a very thin plate, perpendicular to the flow. Most sharp-crested weirs are less than 0.25 inches thick, and many are about 0.10 inches thick. The weir's top edge is often chamfered towards the downstream face. Water or wastewater flows over the weir and the flow is directly proportional to the head or height of water over the weir. A side view of an ideal sharp-crested weir is shown in Figure 1. From Figure 1, several important conclusions can be drawn:

- There must be a straight run of water to the weir, usually at least 20 times the total height of the weir head (H).

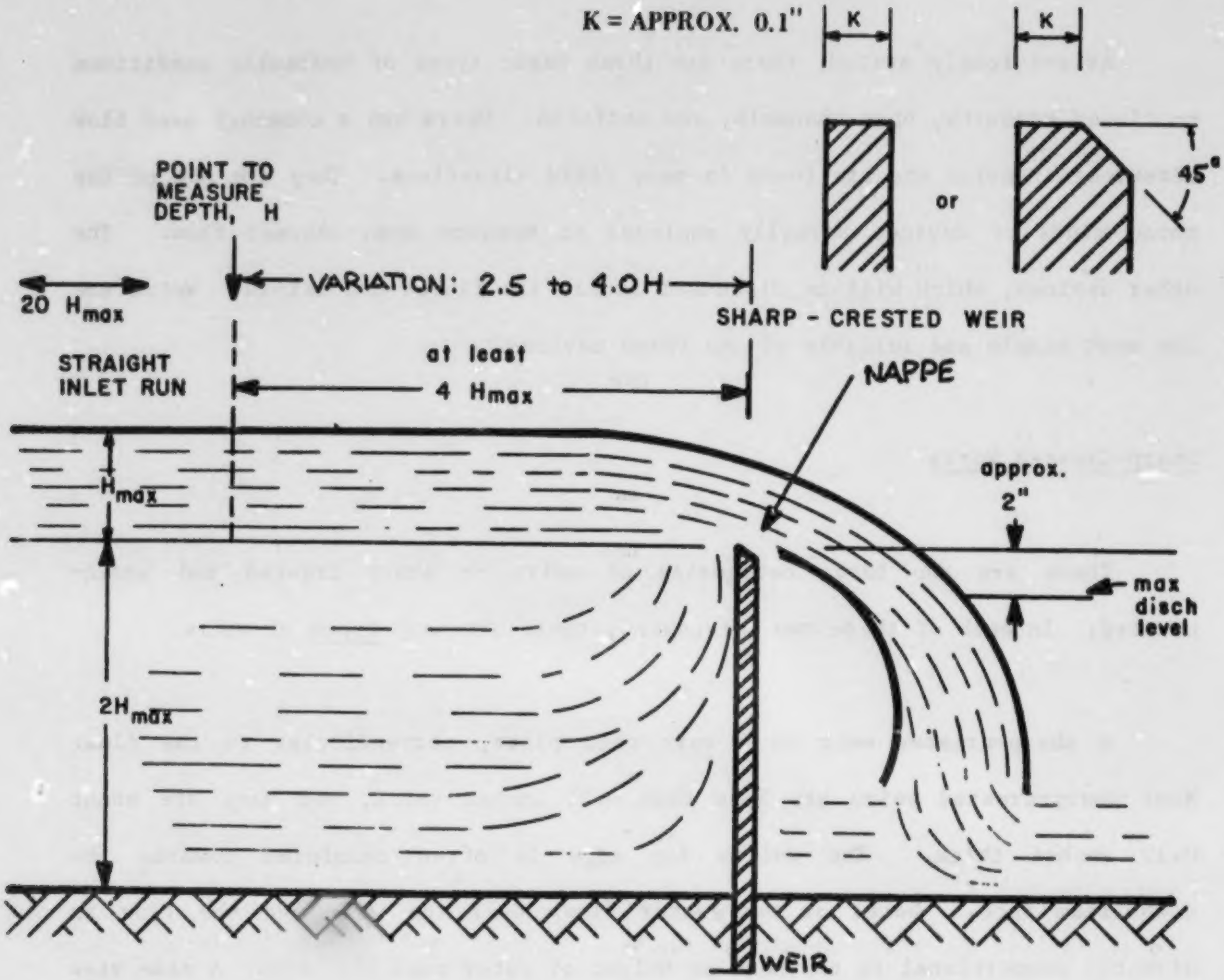

K = APPROX. 0.1"
K
K
or
45°
POINT TO MEASURE DEPTH, H
20 H_{max}
VARIATION: 2.5 to 4.0H
SHARP - CRESTED WEIR
STRAIGHT INLET RUN
at least 4 H_{max}
NAPPE
H_{max}
approx. 2"
max disch level
$2H_{max}$
WEIR

FIGURE 1

SHARP-CRESTED WEIR NOMENCLATURE

(SIDE VIEW)

- The depth of water flow over the weir, or the head, h, must be measured at a distance approximately 4 times H behind the weir.

- The point where water springs freely over the weir is called the nappe.

- The height of the weir must be at least 2 times the maximum head.

K in Figure 1 is the width of the weir itself. The variable, X, is the height of the weir to the bottom of the notch, and H is the head over the weir. This terminology will be used as much as possible from here on. Deviations will be explained to conform with standard nomenclature.

V-Notch Weirs

The most common kind of sharp crested weir encountered in the field is the V-notch weir. A standard V-notch weir is shown in Figure 2. Remember, X is often H, and H_{max} is often h_{max}. Theta (θ), the angle of the notch varies, but the standard angles used are 90°, 60°, and 45°. If a primary flow measurement device must be installed on-site by the NPDES inspector, the V-notch weir is a good candidate. A portable adjustable V-notch weir made of plexiglass with a rubber skirt can be obtained.*

---

* N.B. Products, 35 Beulah Road, New Britain, PA

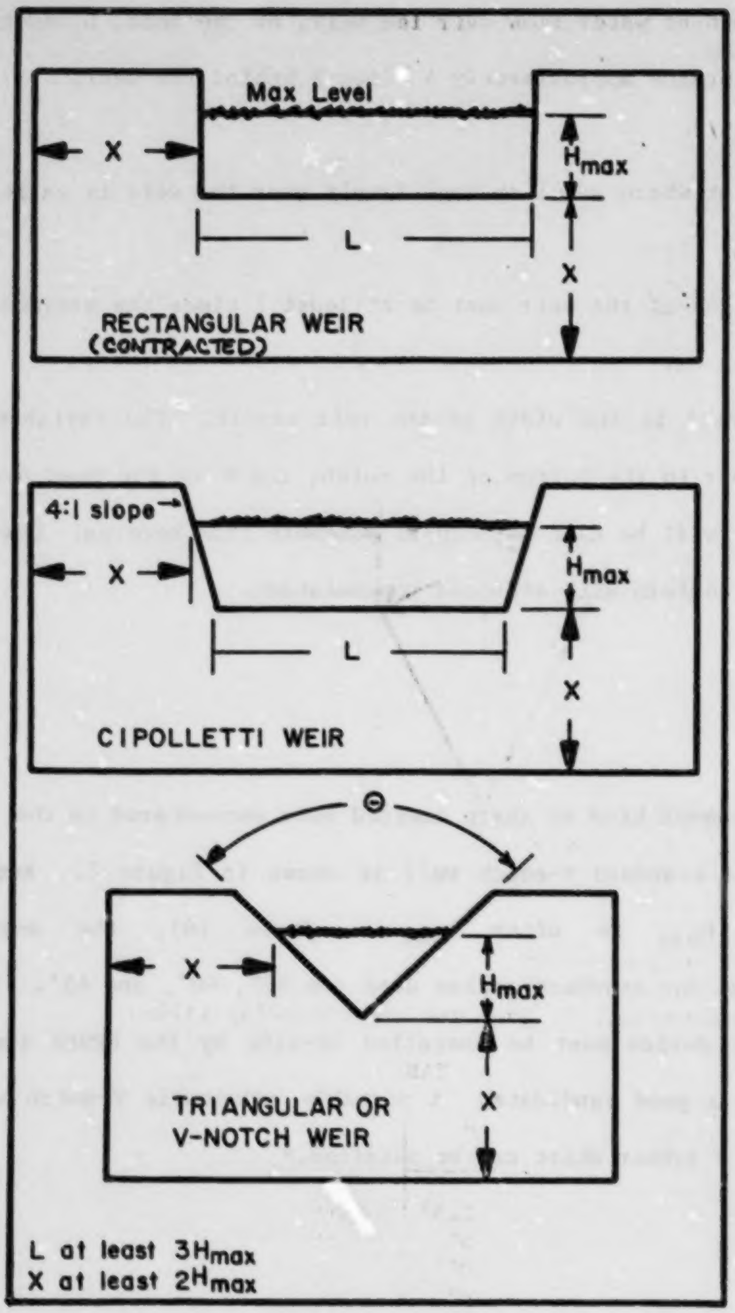

Max Level
X
H max
L
X
RECTANGULAR WEIR
(CONTRACTED)
4:1 slope
X
H max
L
X
CIPOLLETTI WEIR
Θ
X
H max
X
TRIANGULAR OR
V-NOTCH WEIR
L at least 3H max
X at least 2H max

FIGURE 2

THREE COMMON TYPES OF SHARP-CRESTED WEIRS

The formula for the V-notch weir is:[7]

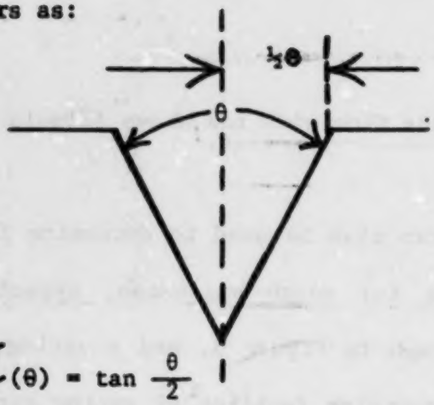
$$Q = CH^{5/2} = CH^{2.5}$$

where: Q = flow (cfs)

C = weir constant

H = head over the weir (ft)

If the notch appears as:



and: $C' = f(\theta) = \tan\frac{\theta}{2}$

and: $C = 2.5C'$

then: $Q = 2.5(\tan\frac{\theta}{2})H^{5/2}$

To find the weir constant, C, just use the following table:

TABLE 3

θ	C
22.5°	0.497
45°	1.035
60°	1.443
90°	2.50

The C for a 90° V-notch weir is 2.5, or the same as the value in the weir formula, because the tangent of $90°/2 = 45°$ is 1.0.!

So the NPDES inspector must:

(1) Measure the V-notch angle, θ, or find it from plant records,

(2) Measure the height or head over the weir, H, usually with a measuring stick,

(3) Then find the instantaneous flow with the above formula for V-notch weirs.

Nomographs and rating curves can also be used to determine flow. On-site, they are easier and faster to use for rough estimates, especially under adverse weather conditions. A nomograph is Figure 3, and a rating curve is Figure 4. An excellent reference which contains families of rating curves may be obtained from the Chicago Pump Company.[8]

Rectangular Weirs

A standard rectangular sharp-crested weir front view is shown in Figure 2. There are two types of rectangular weirs -- contracted and suppressed. When approach conditions allow complete contractions at the ends and at the bottom (the free-fall of the weir head is completely within the notch area), the weir is called a contracted weir. If a rectangular weir is in a flume or channel so the sides of the flume are the ends of the weir; there are no side contractions,

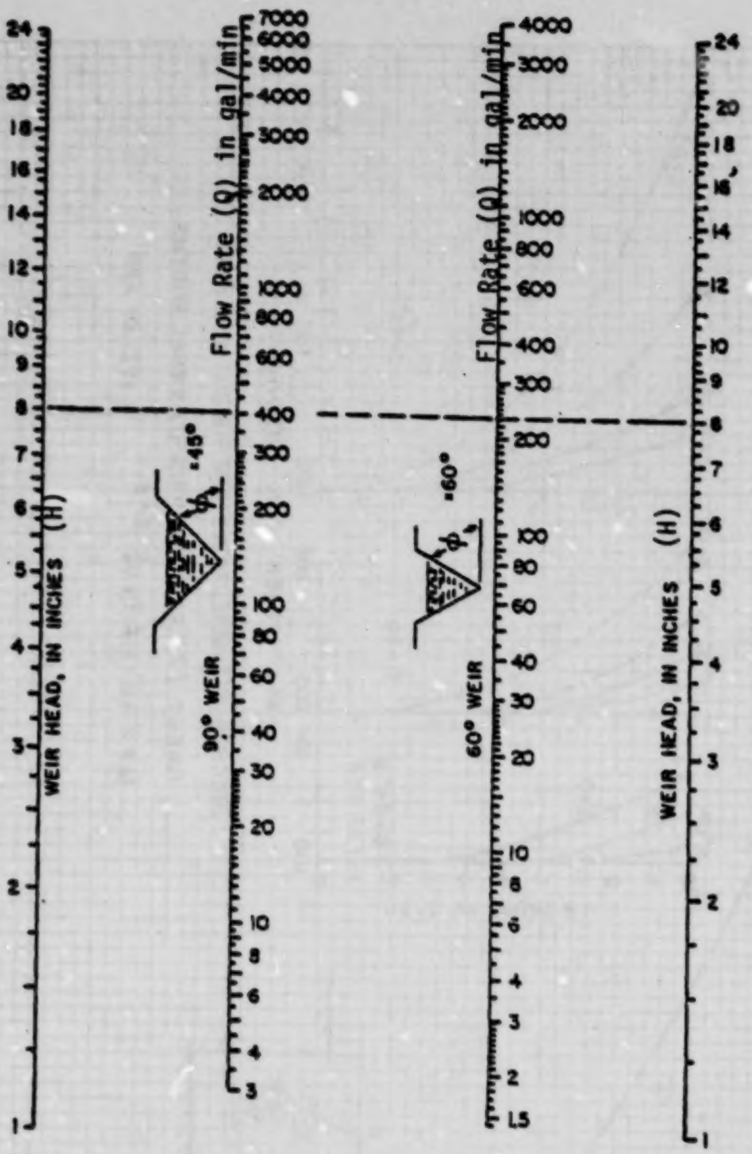


FIGURE 3

FLOW RATES FOR 60° and 90° V-NOTCH WEIRS

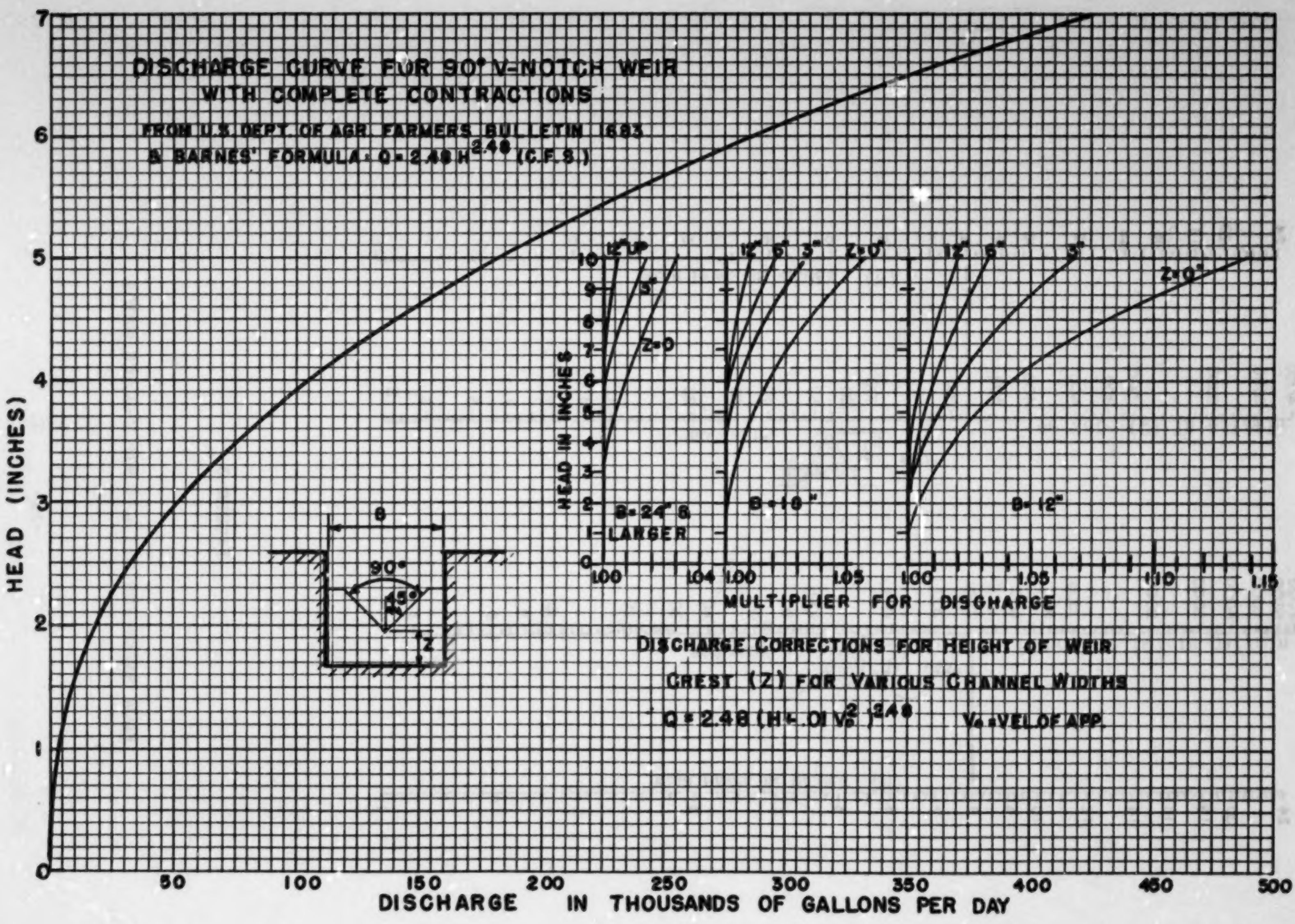

DISCHARGE CURVE FOR 90° V-NOTCH WEIR
WITH COMPLETE CONTRACTIONS
FROM U.S. DEPT. OF AGR. FARMERS BULLETIN 1683
& BARNES' FORMULA: Q = 2.48 H^2.48 (C.F.S.)
HEAD (INCHES)
0
1
2
3
4
5
6
7
DISCHARGE IN THOUSANDS OF GALLONS PER DAY
50
100
150
200
250
300
350
400
450
500
B
90°
45°
Z
HEAD IN INCHES
12" UP
3"
Z=0
B=24" & LARGER
12"
6"
3"
Z=0"
B=18"
12"
6"
3"
Z=0"
B=12"
1.00
1.04
1.00
1.05
1.00
1.05
1.10
1.15
MULTIPLIER FOR DISCHARGE
DISCHARGE CORRECTIONS FOR HEIGHT OF WEIR
CREST (Z) FOR VARIOUS CHANNEL WIDTHS
Q = 2.48 (H + .01 Va²)^2.48
Va = VEL. OF APP.

FIGURE 4

and the nappe does not contract from the width of the channel, this second type of rectangular weir is a called suppressed weir.[9] A contracted rectangular weir is shown as Figure 2, and a suppressed rectangular weir is Figure 5.

The formula for a rectangular weir is:

$$Q = CLH^{3/2}$$

where:

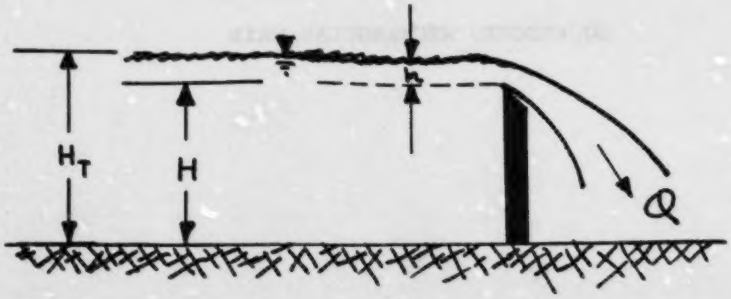
Q = flow (cfs)
C = weir constant
H = head or height of water over weir (ft)
L = weir length (ft)

However, if the velocity of the water approaching the weir is great probably because the length of the cross-section over which the flow falls is large, it is best practice to correct for the approach velocity by using the equation developed by Francis in 1823:[10]

$$Q = 3.33\ (L - 0.1nh)h^{3/2}$$

where:

Q = discharge (cfs)
L = weir length (ft)
n = number of end contractions
h = head = $H_T - H$, where H = normal head level



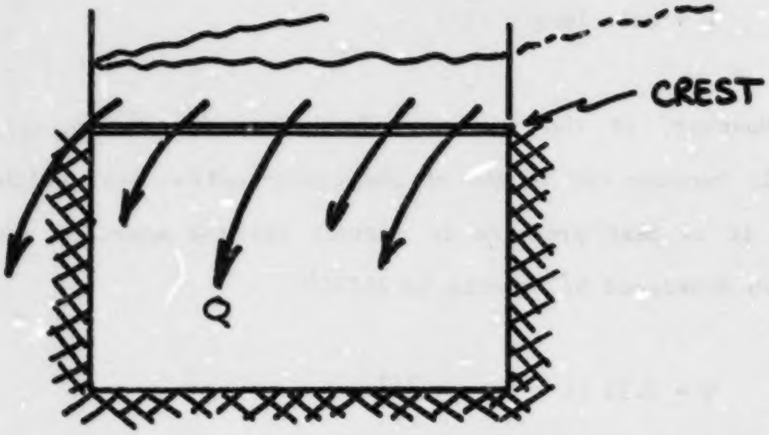


FIGURE 5

SUPPRESSED RECTANGULAR WEIR

In reality,

$$Q = 3.33(L - 0.1\ nH)H^{3/2}$$

For a suppressed weir (see Figure 5):

$$n = 0$$

Thus, $Q = 3.33LH^{3/2}$

For a contracted weir (see Figure 2):

$$n = 2$$

Thus, $Q = 3.33(1 - 0.1\ (2)H)\ H^{3/2}$

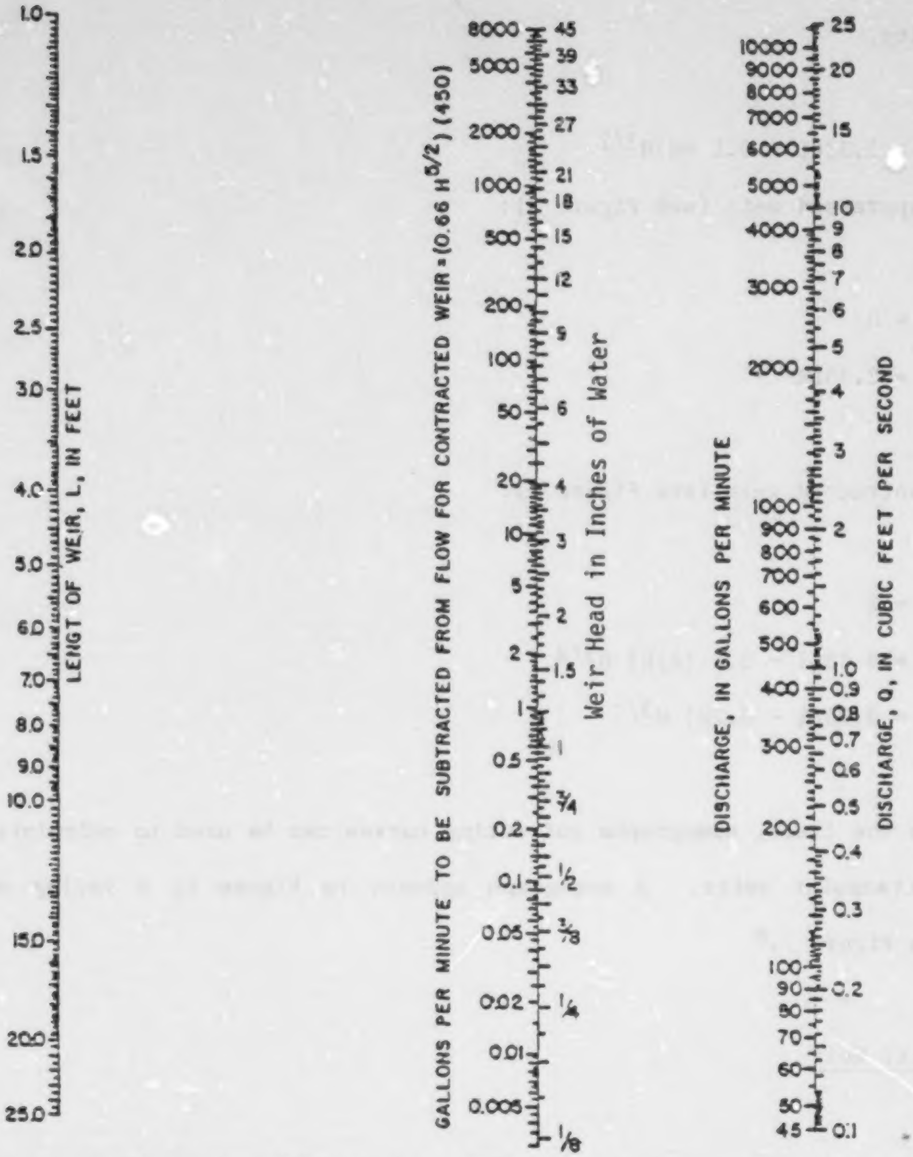
$$Q = 3.33(1 - 0.2H)\ H^{3/2}$$

In the field, nomographs and rating curves can be used to calculate flows over rectangular weirs. A nomograph appears in Figure 6; a rating curve is shown in Figure 7.[8]

Cipolletti Weir

Efforts to compensate for problems with end contractions causing increased turbulence in the flow over the weir led to the development of the Cipolletti weir in 1894.[7,11] The Cipolletti weir is shown in Figure 2. Its formula is:

$$Q = f(Z)$$



Note: based on Francis weir formula as follows:

$Q = 3.33\,LH^{3/2}$ (for suppressed weir)

or

$Q = 3.33\,(L-0.2H)\,H^{3/2}$

$= 3.33\,LH^{3/2} - 0.66\,H^{5/2}$ (for contracted weir with two end contractions)

Where:

Q = discharge, in cubic feet per second.

L = length of weir, in feet.

H = head in feet.

FIGURE 6

NOMOGRAPH FOR CAPACITY OF RECTANGULAR WEIRS

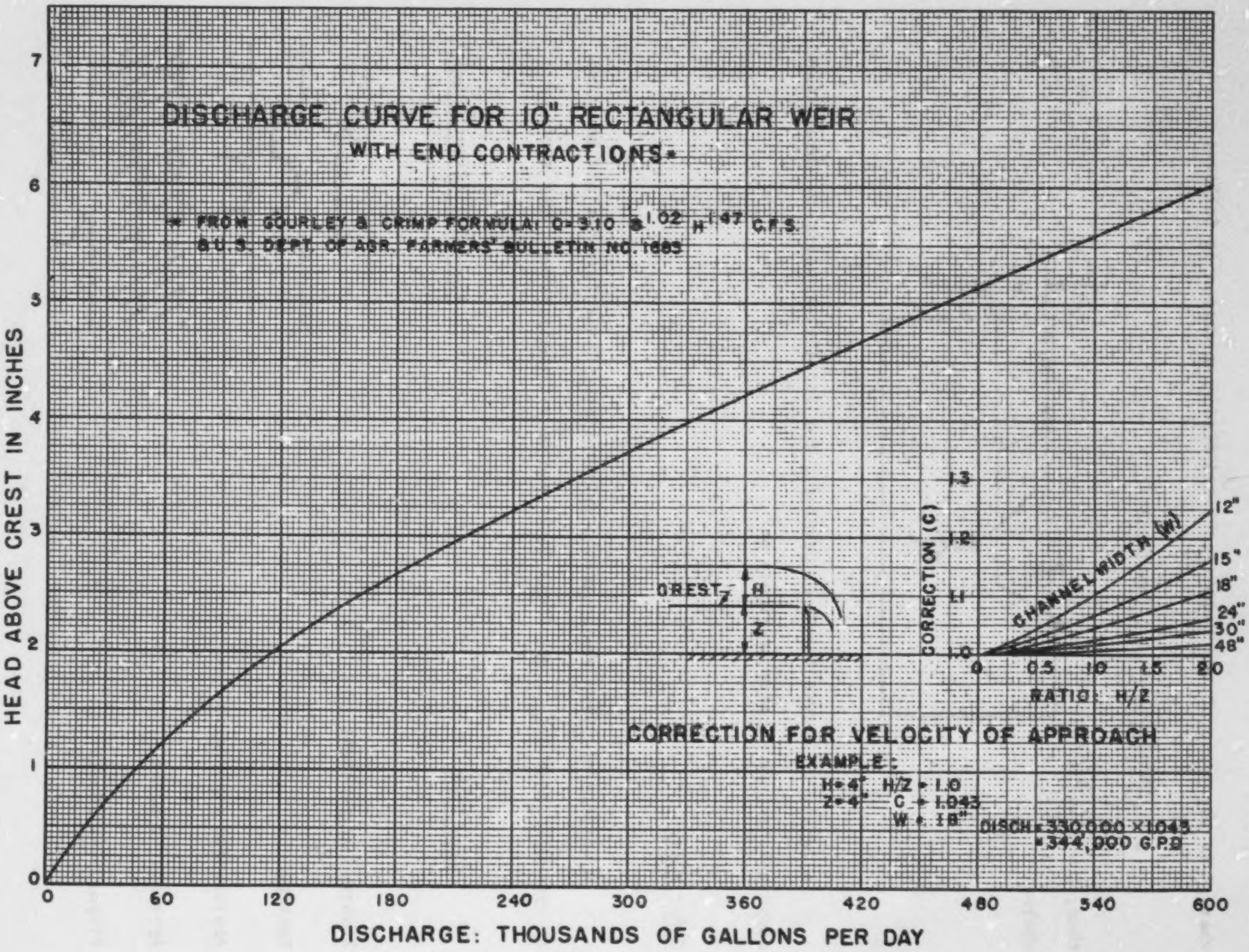

DISCHARGE CURVE FOR 10" RECTANGULAR WEIR
WITH END CONTRACTIONS*
* FROM GOURLEY & CRIMP FORMULA: Q=3.10 B^1.02 H^1.47 C.F.S.
& U.S. DEPT. OF AGR. FARMERS' BULLETIN NO. 1683
HEAD ABOVE CREST IN INCHES
0
1
2
3
4
5
6
7
DISCHARGE: THOUSANDS OF GALLONS PER DAY
0
60
120
180
240
300
360
420
480
540
600
CREST
H
Z
CORRECTION (C)
1.0
1.1
1.2
1.3
CHANNEL WIDTH (W)
12"
15"
18"
24"
30"
48"
0
0.5
1.0
1.5
2.0
RATIO: H/Z
CORRECTION FOR VELOCITY OF APPROACH
EXAMPLE:
H = 4" H/Z = 1.0
Z = 4" C = 1.043
W = 18"
DISCH. = 330,000 × 1.043
= 344,000 G.P.D.

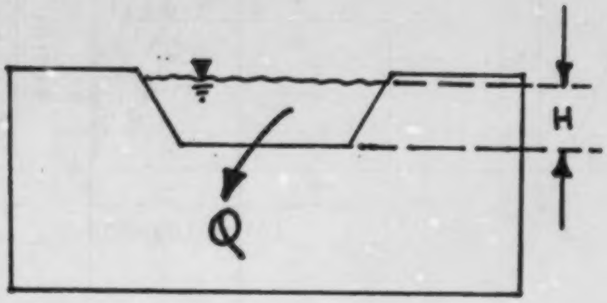
FIGURE 7

where Z is the ratio of the slope of the modified contraction as follows:

$$Z = \frac{\text{vertical}}{\text{horizontal}} = 4:1 \text{ (optimum)}$$

thus, $Q = (3.367)LH^{3/2}$

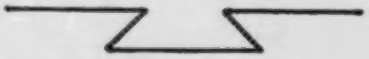
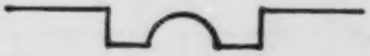
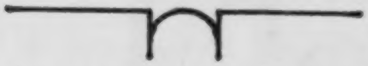
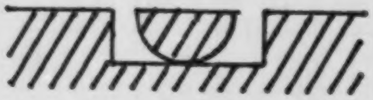
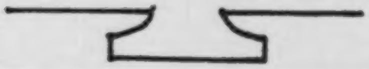
where,



A family of curves has been developed for Cipolletti weirs to ease field computations. It appears as Figure 8.[8]

Other Weirs

A variety of other sharp-crested weirs exist. Their formulas can be found in hydraulic handbooks[7], but their field use is extremely rare. They are:

Weir Type	Pictorial (Front View)
Inverted Trapezoidal	
Poebing	
Approximate Exponential	
Approximate Linear	
Proportional/Sutro	

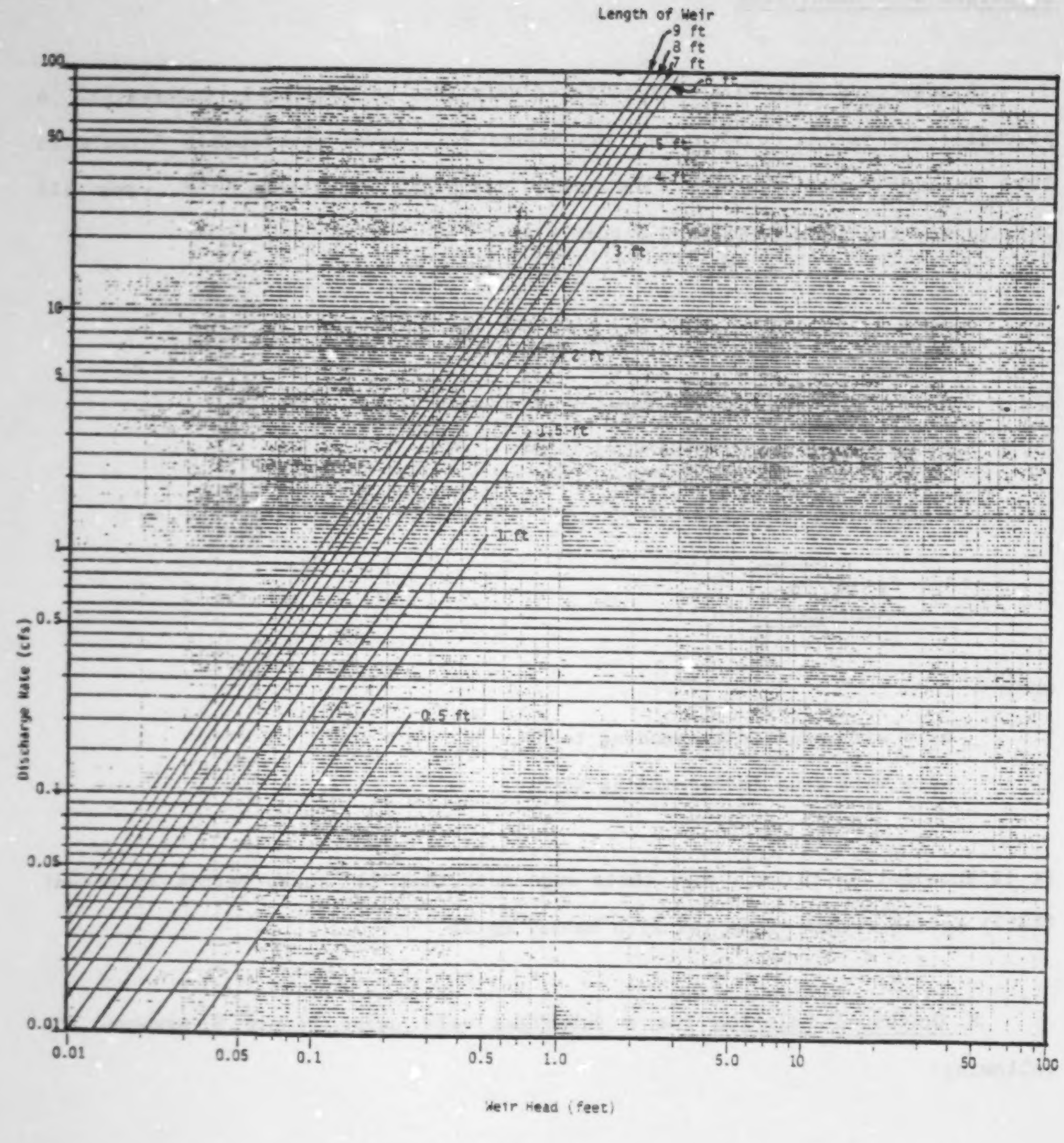


FIGURE 8

DISCHARGE RATE VS WEIR HEAD FOR CIPOLLETTI WEIRS

Submerged Weir Conditions

A submerged weir is a sharp-crested weir that is completely underwater. A pictorial and rating curve appears as Figure 9.[7] Mavis developed a formula in 1949 by which to calculate flow under submerged conditions.[7,12] For all weir types, the formula is (see Figure 9):

$$\frac{Q}{Q_1} = 1 - \left(0.45S + \frac{0.40}{2^{(10-10S)}}\right)$$

where: Q = Discharge for submerged condition in cfs

Q_1 = Free discharge ($H_2 < 0$) in cfs

$$S = \frac{a_2 \sqrt{H_2}}{a_1 \sqrt{H_1}}$$

where:

a_2 = weir area corresponding to H_2

a_1 = weir area corresponding to H_1

It is interesting to note that tests made more than 230 years before by Poleni (1717) agreed within ±2 to ±4% with Mavis' data!

A Graphical Solution for a submerged weir using Figure 9 assumes the following:

Q_1 = discharge at H_1, computed from the equation for the unsubmerged free discharge.

Thus,

$Q_1 = CH_1^n$

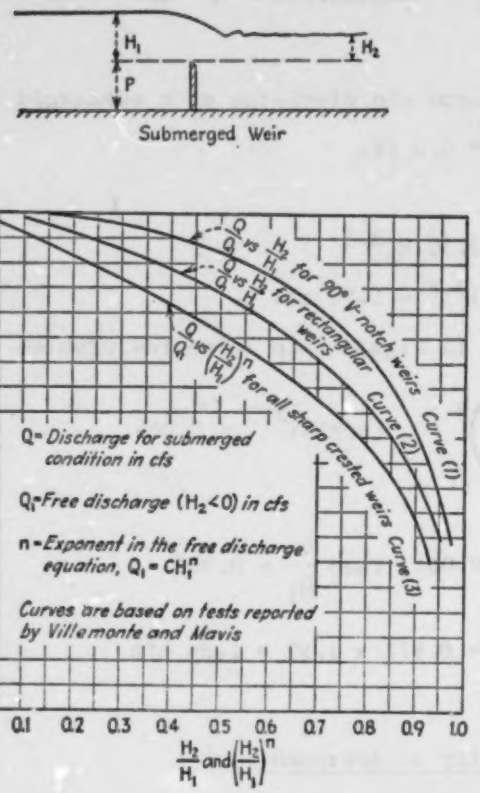


FIGURE 9

SUBMERGED WEIR CALCULATIONS/RATIOS

where:

TABLE 4

Weir Type	n
V-notch	5/2 = 2.5
Rectangular	3/2 = 1.5
Cipolletti	3/2 = 1.5

Example: Determine the discharge of a submerged 90° V-notch weir if $H_1 = 0.9$ ft, and $H_2 = 0.3$ ft.

$$Q_1 = CH_1^n = 2.50\ H_1^{2.5}$$

$$Q_1 = 2.5(0.9)^{2.5} = 1.92 \text{ cfs}$$

Using curve 3 in Figure 9 to solve the problem.

$$\left(\frac{H_2}{H_1}\right)^n = \left(\frac{0.3}{0.9}\right)^{2.5} = (0.333)^{2.5} = 0.064$$

$$\text{If} \left(\frac{H_2}{H_1}\right)^n = 0.064, \text{ then } \frac{Q}{Q_1} = 0.972$$

$$Q = 0.972Q_1 = 0.972 \times 1.92 = 1.86 \text{ cfs}$$

Correcting for Velocity of Approach

As in the case of rectangular weirs, the velocity of approach can become a significant factor in the accuracy of the flow measurement. The NPDES inspector must first remove all trash, slime, and garbage obstructing free-flow over the weir. If quiesent conditions are not met, and if the velocity, V, is much greater than 1 fps (measured commonly with a current meter stuck in front of the weir), it is best to correct for velocity of approach error.

To do this, the most famous hydraulic equation, Darcy's Law or the law of Continuity, is used:

C

V_a	h	$h^{3/2}$	H 0.2	0.4	0.6	0.8	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0
0.4	0.0025	0.0002	1.014	1.007	1.004	1.004	1.004	1.002	1.002	1.002	1.001	1.001	1.001	1.001
.5	.0039	.0003	1.027	1.013	1.009	1.006	1.006	1.004	1.003	1.002	1.002	1.002	1.001	1.001
.6	.0056	.0005	1.037	1.019	1.013	1.009	1.008	1.005	1.004	1.003	1.003	1.002	1.002	1.002
.7	.0076	.0007	1.050	1.026	1.017	1.013	1.011	1.007	1.006	1.004	1.004	1.003	1.003	1.002
.8	.0099	.0010	1.064	1.033	1.022	1.016	1.014	1.009	1.007	1.006	1.005	1.004	1.003	1.003
.9	.0126	.0014	1.062	1.042	1.029	1.021	1.018	1.012	1.009	1.007	1.006	1.005	1.005	1.004
1.0	.0155	.0019	1.098	1.051	1.034	1.027	1.022	1.015	1.011	1.009	1.007	1.006	1.005	1.005
1.1	.0188	.0025	1.122	1.052	1.041	1.031	1.026	1.017	1.013	1.011	1.009	1.008	1.007	1.006
1.2	.0224	.0033	1.141	1.072	1.049	1.037	1.0[illegible]	1.021	1.016	1.013	1.011	1.009	1.008	1.007
1.3	.0263	.0041	1.163	1.084	1.057	1.043	1.036	1.024	1.018	1.015	1.012	1.011	1.009	1.008
1.4	.0305	.0051	1.186	1.096	1.036	1.050	1.041	1.028	1.021	1.017	1.014	1.012	1.011	1.010
1.5	.0350	.0064	1.208	1.109	1.075	1.057	1.047	1.032	1.024	1.019	1.016	1.014	1.012	1.011
1.6	.0398	.0079	1.225	1.122	1.084	1.065	1.052	1.035	1.027	1.022	1.018	1.016	1.014	1.012
1.7	.0449	.0095	1.254	1.135	1.093	1.071	1.059	1.040	1.031	1.025	1.021	1.018	1.016	1.014
1.8	.0504	.0111	1.277	1.149	1.104	1.080	1.065	1.045	1.034	1.027	1.023	1.020	1.017	1.016
1.9	.0561	.0132	1.308	1.165	1.115	1.089	1.072	1.049	1.038	1.030	1.026	1.022	1.019	1.017
2.0	.0622	.0154	1.335	1.181	1.126	1.097	1.079	1.055	1.042	1.034	1.028	1.025	1.021	1.019
2.1	.0686	.0179	1.363	1.197	1.137	1.106	1.087	1.060	1.046	1.037	1.031	1.027	1.024	1.021
2.2	.0752	.0206	1.391	1.213	1.149	1.118	1.094	1.065	1.050	1.039	1.034	1.029	1.026	1.023
2.3	.0822	.0235	1.420	1.231	1.161	1.124	1.102	1.071	1.054	1.044	1.047	1.032	1.028	1.025
2.4	.0895	.0268	1.449	1.248	1.176	1.134	1.110	1.077	1.059	1.047	1.040	1.034	1.030	1.027
2.5	.0972	.0303	1.480	1.266	1.187	1.145	1.119	1.083	1.063	1.051	1.043	1.037	1.033	1.029
2.6	.1051	.0340	1.511	1.285	1.200	1.155	1.128	1.088	1.068	1.055	1.046	1.040	1.035	1.032
2.7	.1133	.0381	1.542	1.303	1.213	1.166	1.137	1.095	1.073	1.059	1.050	1.043	1.038	1.034
2.8	.1219	.0426	1.573	1.322	1.228	1.178	1.146	1.100	1.078	1.063	1.053	1.046	1.041	1.036
2.9	.1307	.0472	1.606	1.341	1.242	1.189	1.155	1.108	1.083	1.067	1.057	1.049	1.043	1.039
3.0	.1399	.0524	1.637	1.361	1.256	1.199	1.165	1.115	1.088	1.072	1.061	1.053	1.046	1.041

TABLE 5

TABLE FOR CALCULATING VELOCITY OF APPROACH FOR A SHARP-CRESTED WEIR

$$Q = VA$$

where:

Q = flow (cfs)

V = velocity (fps)

A = area (ft^2 or square ft)

Modified, Darcy's Law becomes:

$$V_a = Q/A$$

where V_a is the velocity of approach. From Table 5, enter V_a (obtained with the current meter), and H (measured), and read $\mathbb{C}$.[3] Thus,

$$Q_a = \mathbb{C}Q$$

where:

Q_a = flow corrected for V_a, cfs.

$\mathbb{C}$ = constant from Table 5.

Q = flow calculated by the appropriate weir formula, cfs.

Weir Inspections

For a sharp-crested weir, NPDES inspectors should note the following during the inspection:[13]

- Is the plate perpendicular? Are the sides vertical in the channel?

- Is the plate thickness about 0.1"? Does it have a 45° chamfer (see Figure 1)?
- Is $P + H_T > 2H$? Is $P + H_T \nless 1$ ft.?*
- Does the nappe touch the upstream side? Does air circulate freely underneath on the downstream side?
- Is H measured at about $4H_{max}$ upstream from the weir?
- Is the channel straight and long upstream from the weir?
- Has V_a been corrected for?
- Do the sides and bottom of the weir allow leaks? There should be none.

Broad-Crested Weirs

A broad-crested weir, from the side view, is much wider than a sharp-crested weir. Lateral widths can even exceed 10 feet. There is neither a notch nor nappe, and the weir shapes are very different. The weir notch is mounted in a wall too thick for the water to spring past. Broad-crested weirs do not have many advantages over sharp-crested weirs except they are relatively free from fouling by suspended solids deposition on their leading edge. Broad-crested weirs usually extend from bank to bank unlike sharp-crested weirs which have their "working" section, or the notch, at mid-point in the channel (except in the case of suppressed sharp-crested weirs which will be discussed later as a special case).

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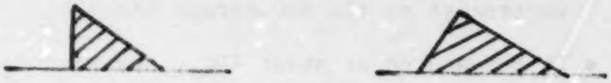
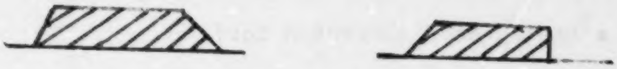
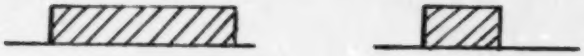
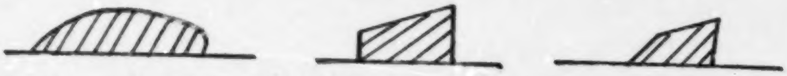
* $\nless$ is "not less than"

There are four types of broad-crested weirs, named for their side-view shapes. They are:

Types	Shapes
• Triangular	
• Trapezoidal	
• Rectangular	
• Irregular	

In these cases the flow over the weir is from the left to the right of the page, in this direction: $Q \longrightarrow$

The general broad-crested weir formula is:

$$Q = CLH^{3/2}$$

C, or the weir coefficient, can be found in any good hydraulics handbook.[7]

FLUMES

Flumes are commonly used to measure water or wastewater flow. They can be used alone, or in combination with some other flow measurement device (see Figure 10).[9] Flumes are usually specially shaped open channel flow sections installed in canals, laterals, ditches or at the end of unit processes to measure flows.

Parshall Flumes

The NPDES inspector will often encounter Parshall Flumes, especially if he is inspecting Publicly-Owned Treatment Works (POTWs). A standard Parshall flume, originally developed in 1922 and tested in the Arkansas River Valley between 1926 and 1930,[14] is shown in Figure 11.[3]

The Parshall flume uses the venturi principle of flow restriction. It has the following advantages and disadvantages:[9]

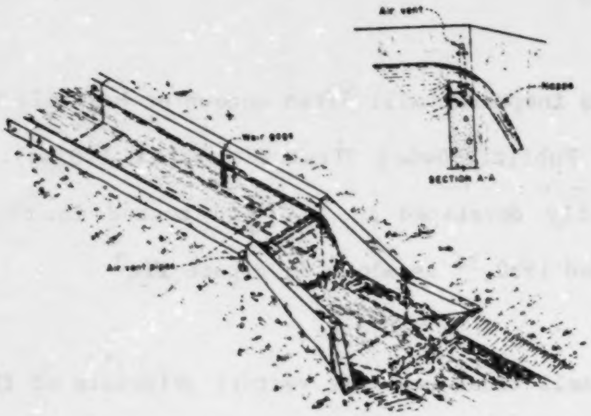

Air vent
Nappe
SECTION A-A
Weir gage
Air vent pipes

FIGURE 10

TYPICAL SUPPRESSED WEIR IN A FLUME DROP

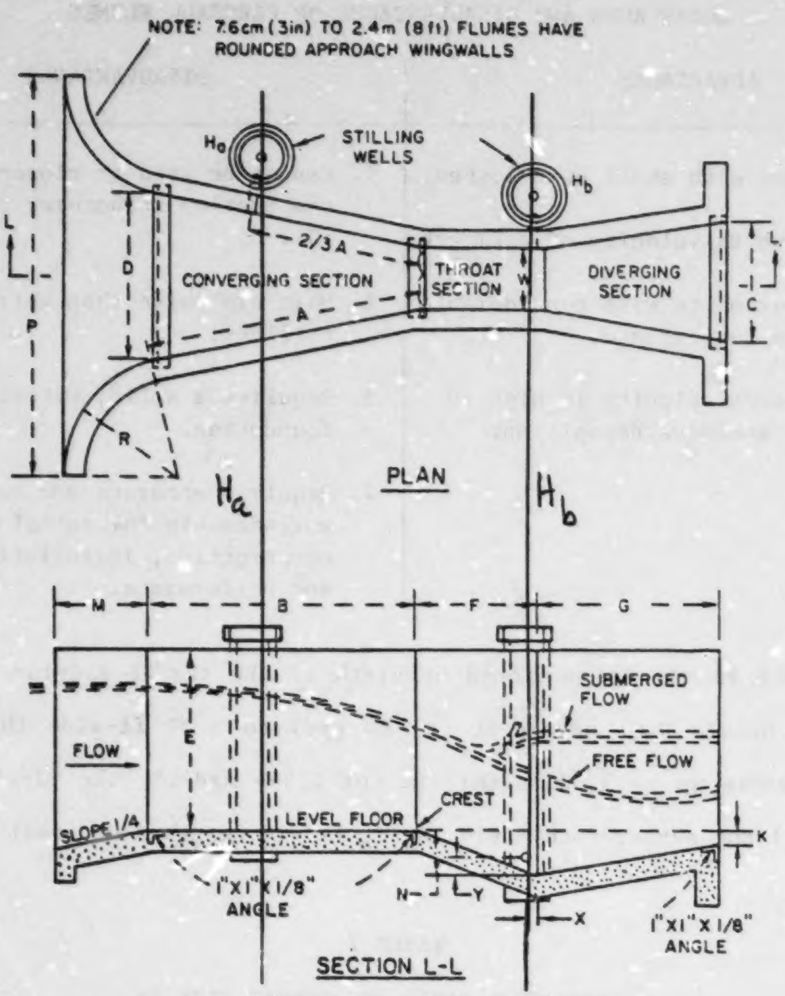

NOTE: 7.6cm (3in) TO 2.4m (8 ft) FLUMES HAVE ROUNDED APPROACH WINGWALLS
STILLING WELLS
Ha
Hb
L
D
P
R
2/3 A
A
CONVERGING SECTION
THROAT SECTION
W
DIVERGING SECTION
L
C
PLAN
Ha
Hb
M
B
F
G
E
FLOW
SUBMERGED FLOW
FREE FLOW
SLOPE 1/4
LEVEL FLOOR
CREST
K
1"X1"X1/8" ANGLE
N
Y
X
1"X1"X1/8" ANGLE
SECTION L-L

FIGURE 11

CONFIGURATION FOR A STANDARD PARSHALL FLUME

TABLE 6

ADVANTAGES AND DISADVANTAGES OF PARSHALL FLUMES

ADVANTAGES	DISADVANTAGES
1. Can operate with small head losses.	1. Cannot be used in close-coupled, combination structures (See Figure 10).
2. Insensitive to velocity of approach.	2. More expensive than weirs and orifices.
3. Good measurements with considerable downstream submergence.	3. Requires a solid, watertight foundation.
4. The discharge velocity is high to eliminate sediment deposition.	4. Requires accurate and careful worksmanship for satisfactory construction, installation, and performance.

Parshall flumes are designed by width of the throat section. A 1.0 in. section can handle 0.01 second-ft (or 80 gpd) to a 50 ft-wide throated flume which can handle up to 3,000 second-ft (or 1,944 mgd).* The discharge is not reduced until the submergence ratio, H_b/H_a is greater than or equal to:

TABLE 7

SUBMERGENCE RATIO VS. THROAT SIZE IN PARSHALL FLUMES

% SUBMERGENCE	THROAT WIDTH
50	1 - 3 in.
60	6 - 9 in.
70	1 - 8 ft.
80	8 - 50 ft.

---

*1 second-ft = 450 gpm = 0.648 mgd

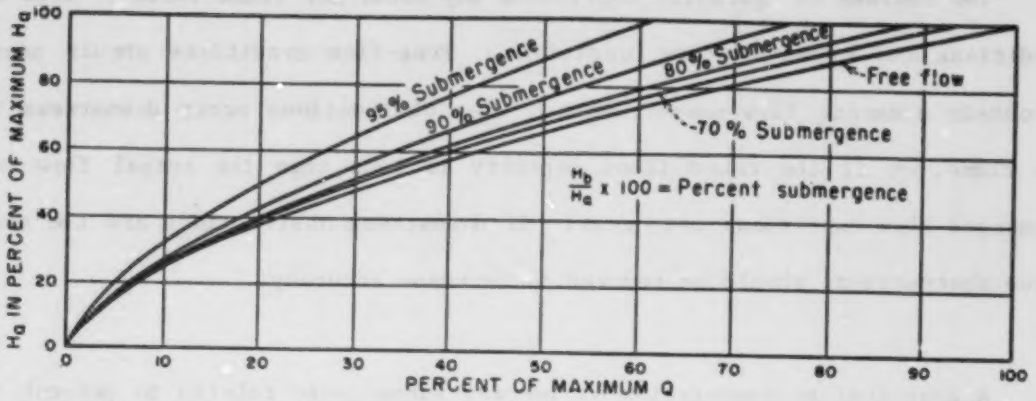
Two regimes of hydraulic conditions may occur for flume flows -- free-flow conditions and submerged-flow conditions. Free-flow conditions should prevail to obtain accurate flow measurements. When obstructions occur downstream from the flume, or if the rated flume capacity is less than the actual flow rate, submerged flow conditions may occur. If downstream obstructions are the cause, these obstructions should be removed to increase accuracy.

A graphical representation of percent submergence related to percent flow appears in Figure 12, along with a nomograph to determine free-flow discharge in 3 inch to 8 foot Parshall flumes.[9] Use Figure 13 to calculate the flow in a submerged Parshall flume.[9]

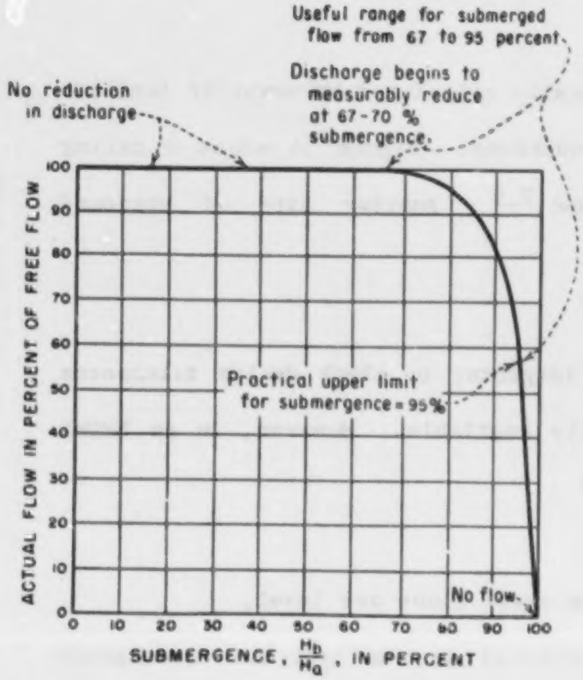
The flow through Parshall Flumes is easily calculated by means of families of rating curves for various hydraulic conditions. Figure 14 shows a rating curve and a picture of a Parshall flume.[7,9] Another type of standard rating curve is shown as Figure 15.[8]

It is not appropriate for the NPDES inspector to check design tolerances of a Parshall flume unless they are readily available. However, in an NPDES inspection, the inspector should see that:[4]

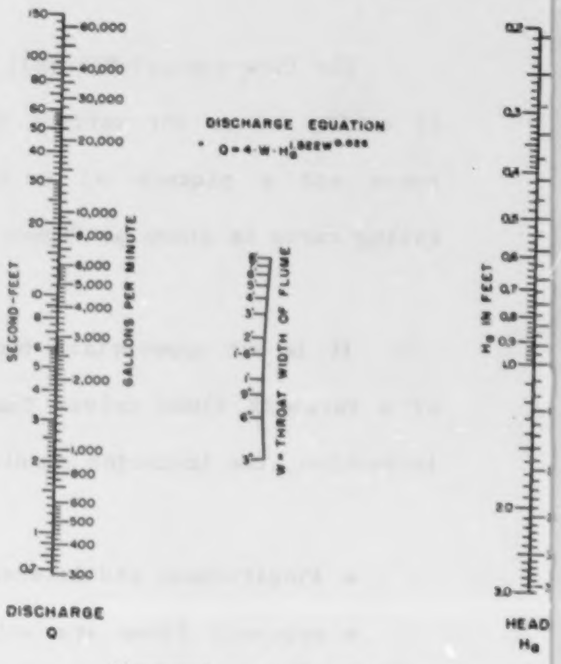
- longitudinal and lateral axes of the crest floor are level,
- approach flows are uniformly distributed in the upstream convergence section,
- head measurement devices (to be discussed later) are installed at the correct location, and
- flow variations are within the range for which the flume is accurate (see Table 7).



Typical discharge curves for Parshall flumes with free flow and with submerged conditions.



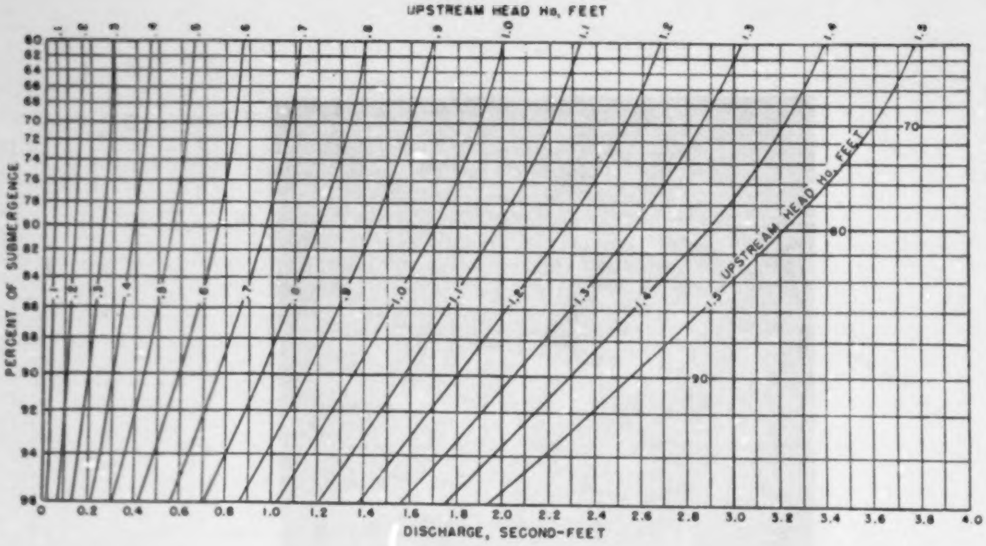
Typical discharge reduction caused by submergence in 1- to 8-foot Parshall flumes.



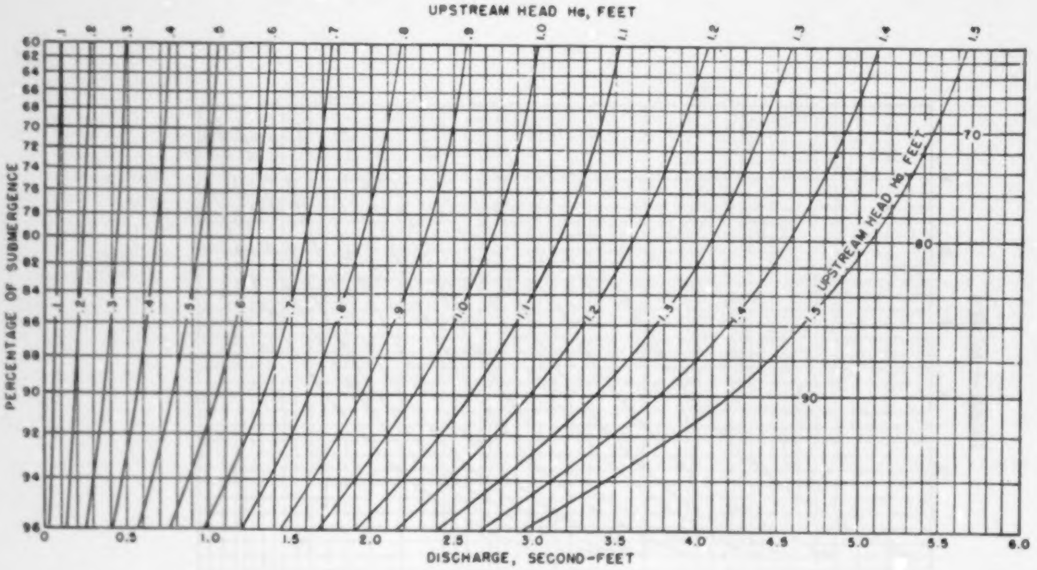
Nomograph for free-flow discharge through 3-inc 8-foot Parshall flumes.

FIGURE 12

PARSHALL FLUME DISCHARGE CURVES



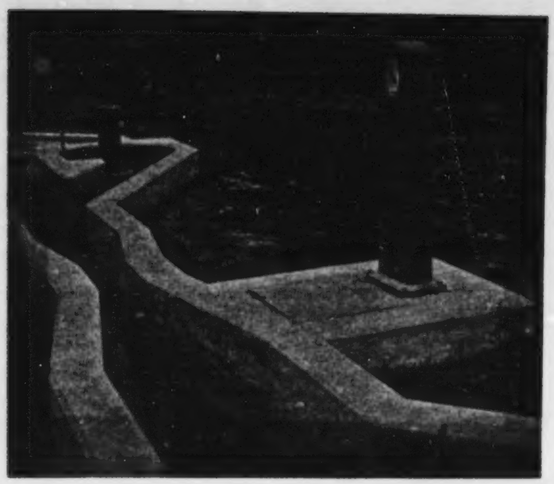
—Diagram for determining rate of submerged flow for a 6-inch Parshall flume. (Courtesy U.S. Soil Conservation Service.)



—Diagram for determining rate of submerged flow for a 9-inch Parshall flume. (Courtesy U.S. Soil Conservation Service.)

FIGURE 13

TYPICAL FLUME SUBMERGENCE FLOW RATE



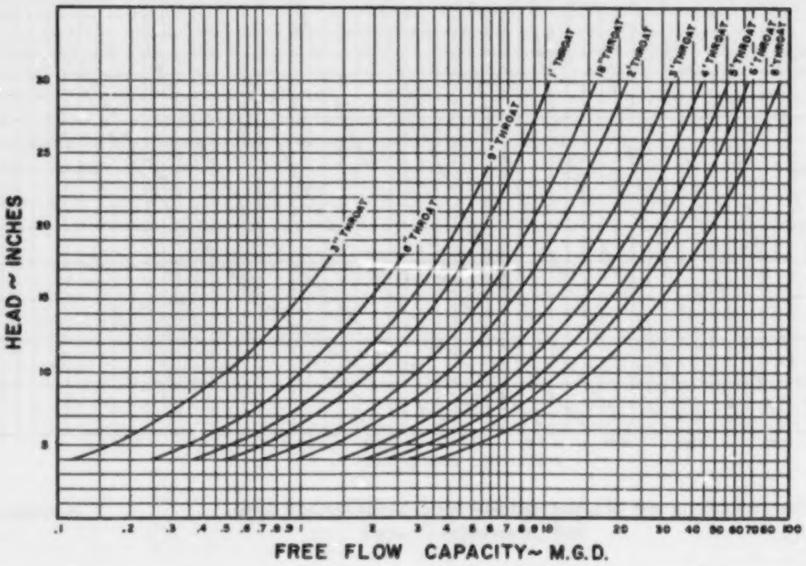


FIGURE 14

PARSHALL FLUMES

TYPICAL INSTALLATION AND CAPACITY CURVES

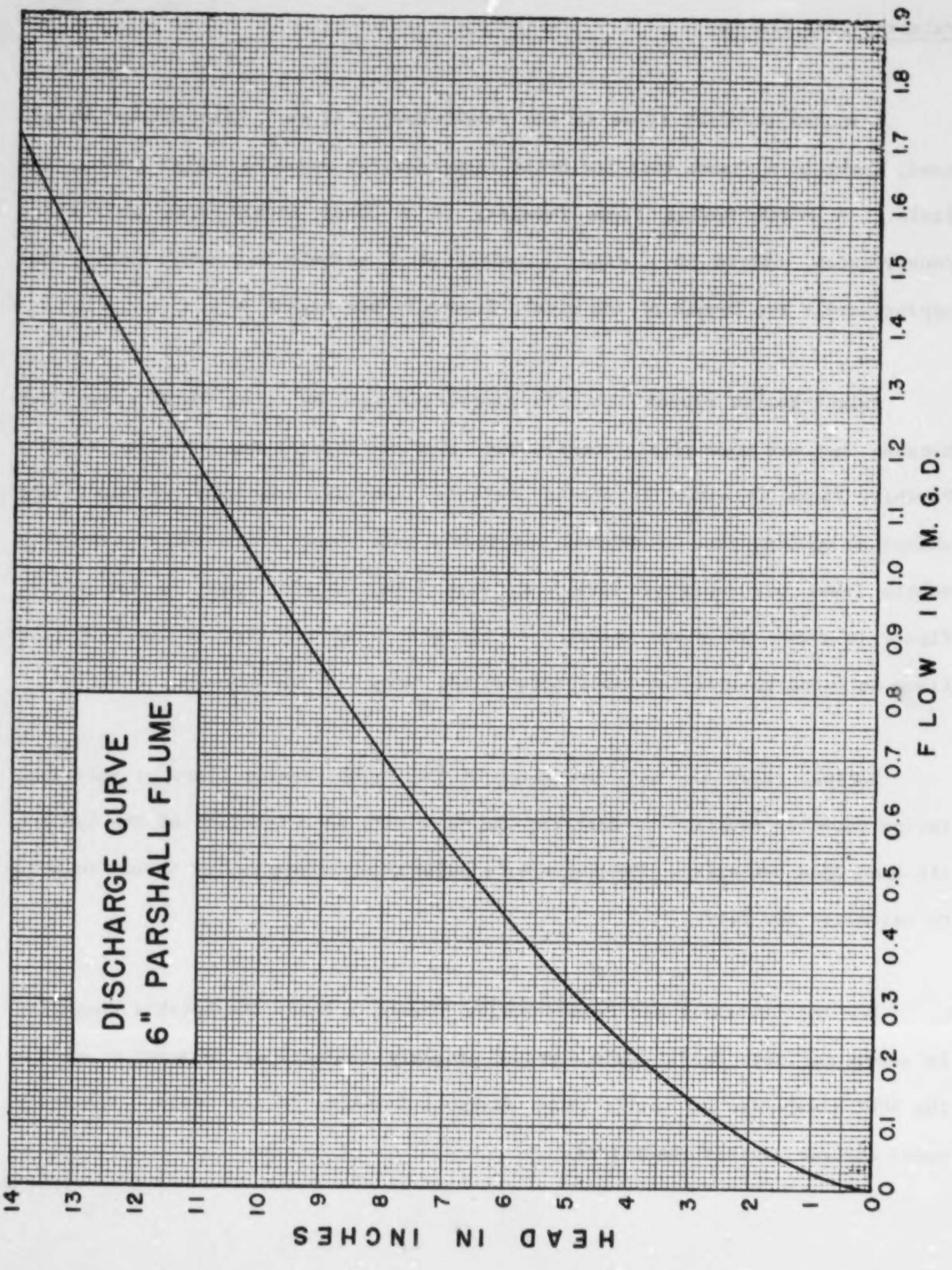

DISCHARGE CURVE
6" PARSHALL FLUME
HEAD IN INCHES
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
FLOW IN M. G. D.
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9

FIGURE 15

Palmer-Bowlus Flumes

A second standard flume is the Palmer-Bowlus flume. Although not widely used, there is a chance that the NPDES inspector may encounter this flume in the field. A Palmer-Bowlus flume consists of a level floor, with or without constriction, placed in a pipe (commonly at a manhole for easy access) for approximately the length of the pipe's diameter (see Figure 16).

Palmer-Bowlus flumes have the same advantages as other flume types, but because they are more simply constructed, they are compact and portable, which a Parshall flume is not. To obtain accurate readings, the Palmer-Bowlus flume should be smooth, free of debris, and level. The flume kit usually contains a bubble level (or bullseye level) to ease leveling the flume in the field. Figure 16 shows the flume inserted in the sewer and the tire accompanying the flume inflated to seal the space between the flume and the sewer.

Because there are many varieties of the Palmer-Bowlus flume, a universal rating curve or equation to describe the flow rate as a function of the hydraulic head does not exist. Manufacturer's calibrated rating curves should be used to calculate the flow.

For both Parshall and Palmer-Bowlus flumes, a handy inspector's checklist is shown as Table 8.[15] Again, it is probably impractical in most cases for the NPDES inspector to verify flume design tolerances in the field, especially under adverse weather conditions.

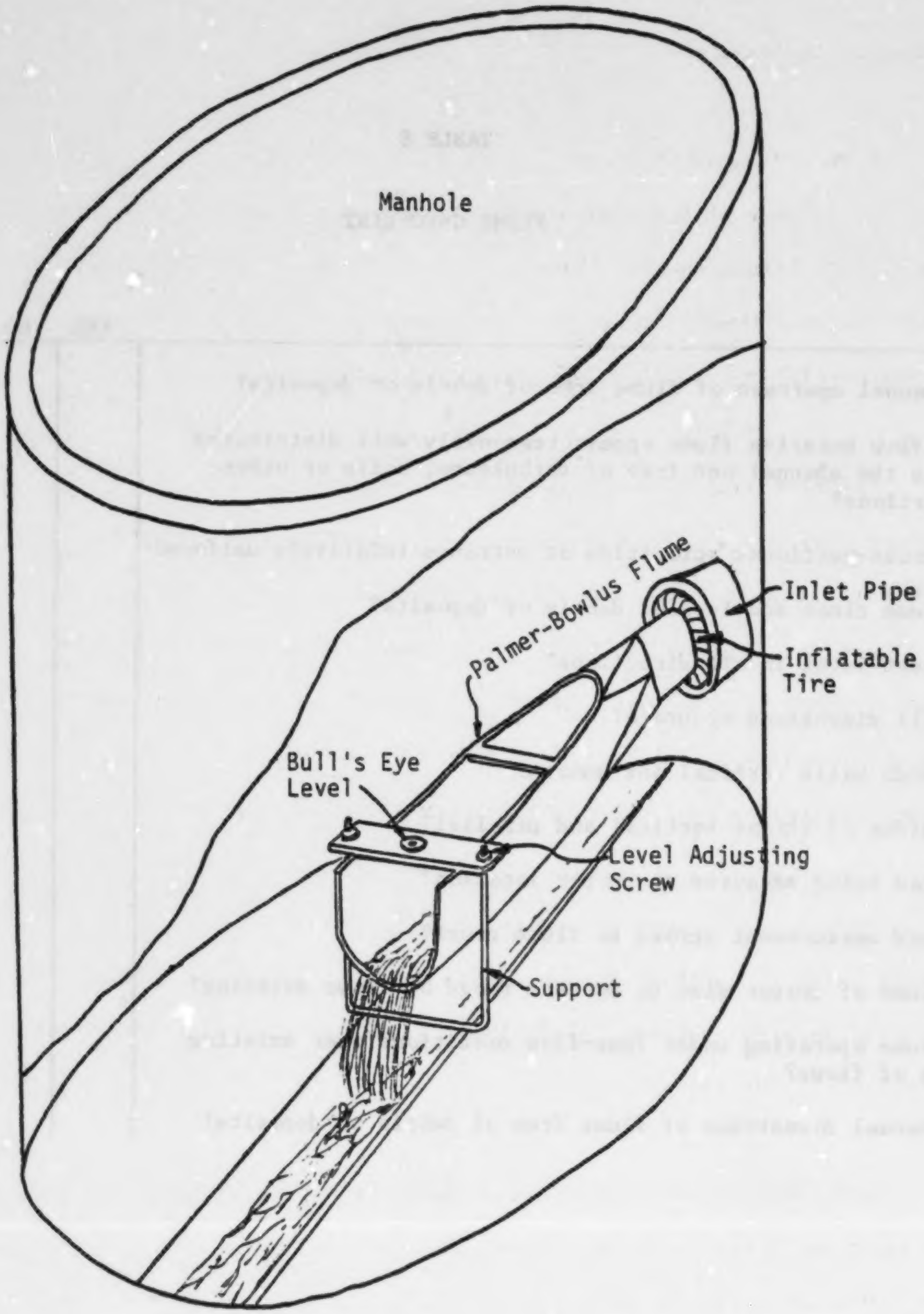

Manhole
Palmer-Bowlus Flume
Inlet Pipe
Inflatable Tire
Bull's Eye Level
Level Adjusting Screw
Support

FIGURE 16

TYPICAL INSTALLATION OF A TEMPORARY FLUME

TABLE 8

FLUME CHECKLIST

	YES	NO	REMARKS
1. Is channel upstream of flume free of debris or deposits?			
2. Does flow entering flume appear reasonably well distributed across the channel and free of turbulence, boils or other distortions?			
3. Are cross-sectional velocities at entrance relatively uniform?			
4. Is flume clean and free of debris or deposits?			
5. Is crest level in all directions?			
6. Are all dimensions accurate?			
7. Are side walls vertical and smooth?			
8. Are sides of throat vertical and parallel?			
9. Is head being measured at proper location?			
10. Is head measurement zeroed to flume crest?			
11. Is flume of proper size to measure range of flows existing?			
12. Is flume operating under free-flow conditions over existing range of flows?			
13. Is channel downstream of flume free of debris or deposits?			

PITOT TUBES

The pitot tube, invented by Henri Pitot in 1732,[1] is a basic device used to measure the velocity at a point in a flowing fluid. It is still one of the most accurate methods of measuring velocity (see Figure 17). Its optimium accuracy is about ±1% of the flow rate.[17] The pitot tube also has the advantage of being insensitive to flow alignment, if the yaw misalignment is less than 15°. Its application is in pipes flowing full, usually under pressure.

Using Bernoulli's equation, the equation for the manometer, and solving for velocity yields the equation for the Pitot tube:[18]

$$v = \sqrt{2g\ R'\left(\frac{S_o}{S} - 1\right)}$$

where:

v = velocity (fps)

g = acceleration of gravity = 32.2 ft/sec^2

R' = total pressure (ft)

S_o = specific gravity of mercury = 13.57

S = specific gravity of water = 1.0

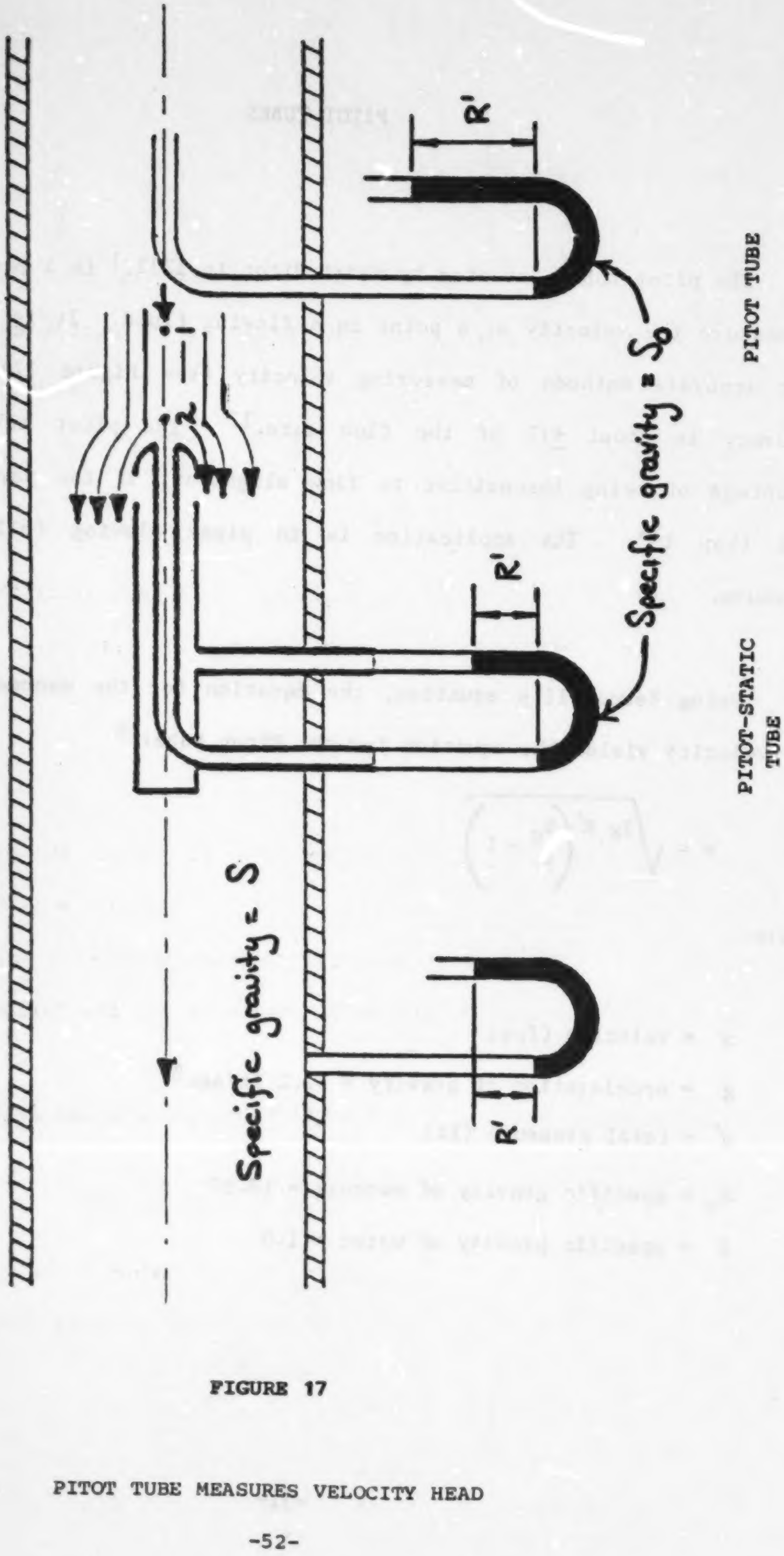

R'
R'
R'
Specific gravity = S
Specific gravity = S_0
PITOT TUBE
PITOT-STATIC TUBE
1
2

FIGURE 17

PITOT TUBE MEASURES VELOCITY HEAD

simplifying:

$$v = 28.45\sqrt{R'}$$

However, because of the uncertainty in the static pressure measurement (from the center of the tube to the pipe wall), a corrective coefficient C must be included:

$$v = 28.45C\sqrt{R'}$$

C must be known or determined by calibration. By substituting the above equation into Darcy's Law, Q = VA, an expression for the flow results:

$$Q = A\left(28.45C\sqrt{R'}\right)$$

But this exercise is not as difficult as it seems. The graphical solution in Figure 18 can be employed to relate velocity head to velocity under certain flow regimes or conditions. Moreover, this discussion will only probably serve as background information for the NPDES inspector for the following reasons:

(1) Because of high suspended solid loads in most wastewaters, pitot tubes are impracticable due to fouling.

(2) Because the NPDES inspector will encounter flows generally in open channels, the pitot tube will not be seen in most field situations.

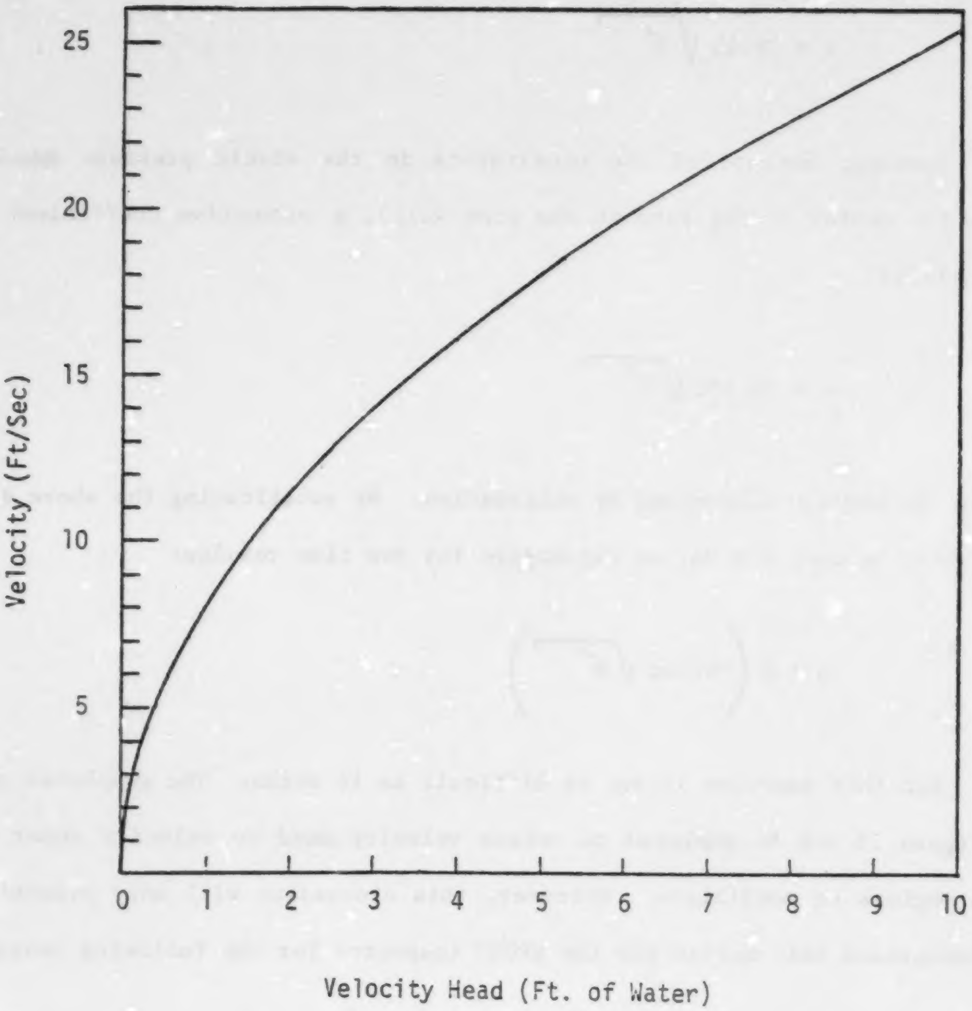


FIGURE 18

GRAPH FOR CONVERTING VELOCITY HEAD TO VELOCITY

FOR PITOT TUBES, VENTURI TUBES, AND ORIFICES

UNDER PRESSURE

(3) Pitot tubes are difficult to maintain under usual field conditions, so their use is limited.

METHODS USED TO MEASURE WATER HEIGHT (HEAD)

There are many methods and types of gauges for measuring water height or head over a weir or flume or in a sewer. All have different types of applications, but the NPDES inspector will probably see or use a variety of them in the field.

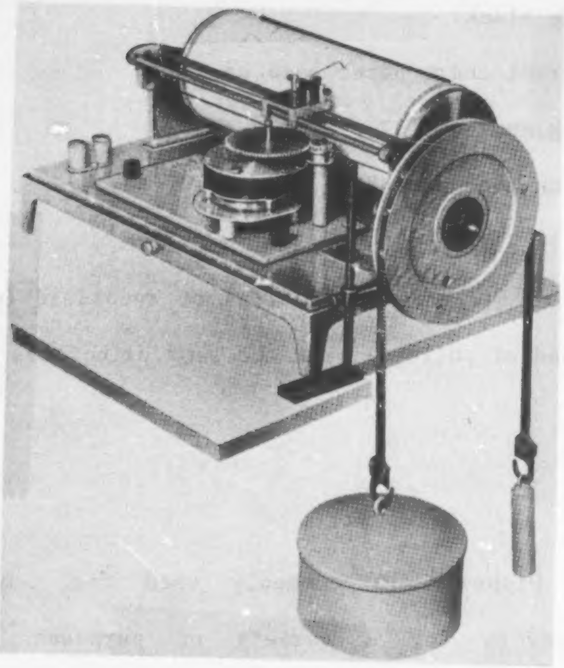
Stevens Meters or Drum Recorders

A very common type of water stage recorder is a horizontal drum-type. This author classifies all such recorders as Stevens meters because that is a well-known trademark. Other product types are available.

The Stevens* recorder is a versatile, horizontal-drum, graphic recorder where the clock positions the pen along the drum axis and the gage height element rotates the drum (see Figure 19). Usually, an 8-day spring-driven clock is used.[9] The Stevens meter or recorder is usually installed in a stilling tube, so that the float can remain level and relatively stable. The NPDES inspector can easily make a stilling tube from PVC pipe about 1/2 to 1 inch larger in diameter than the float with several holes drilled in the pipe below the water line to ease water movement in the tube.

---

*Leupold and Stevens Instruments, Inc.



—Horizontal drum water-stage recorder. The time element records parallel to the axis of the drum. (Courtesy Leupold and Stevens Instruments, Inc.)

FIGURE 19

On an inspection, the inspector must note:

- Is the stilling well perpendicular to the meter?
- Has trash and debris accumulated inside the meter? (If so, clean it out).
- Is the float dirty? Is the float perpendicular to the flow and mounted properly in the stilling well?
- Is the cable slack?
- Has the correct chart paper been used?
- Is there ink in the pen?
- Has the clock been wound?

Of course there are other variations of recorders (e.g., vertical drums, or gear boxes instead of pulleys), but the same principles apply.

Manning Dippers

The Manning Dipper™ is commonly used for temporary, direct flow measurements in sewers for a variety of purposes including compliance monitoring. The Manning Dipper is easy to install, versatile and rugged.[19] The Dipper uses a dipping-probe technique that detects the liquid surface by use of a thin, corrosion-resistant metal probe lowered on a cable controlled by a precision motor. The probe continuously tracks the changes in liquid level with a regulated "dipping" action, normally maintaining its position just above the liquid's surface (see Figure 20).[20] A Dipper installed in a manhole in conjunction with a Palmer-Bowlus flume is shown in Figure 21.

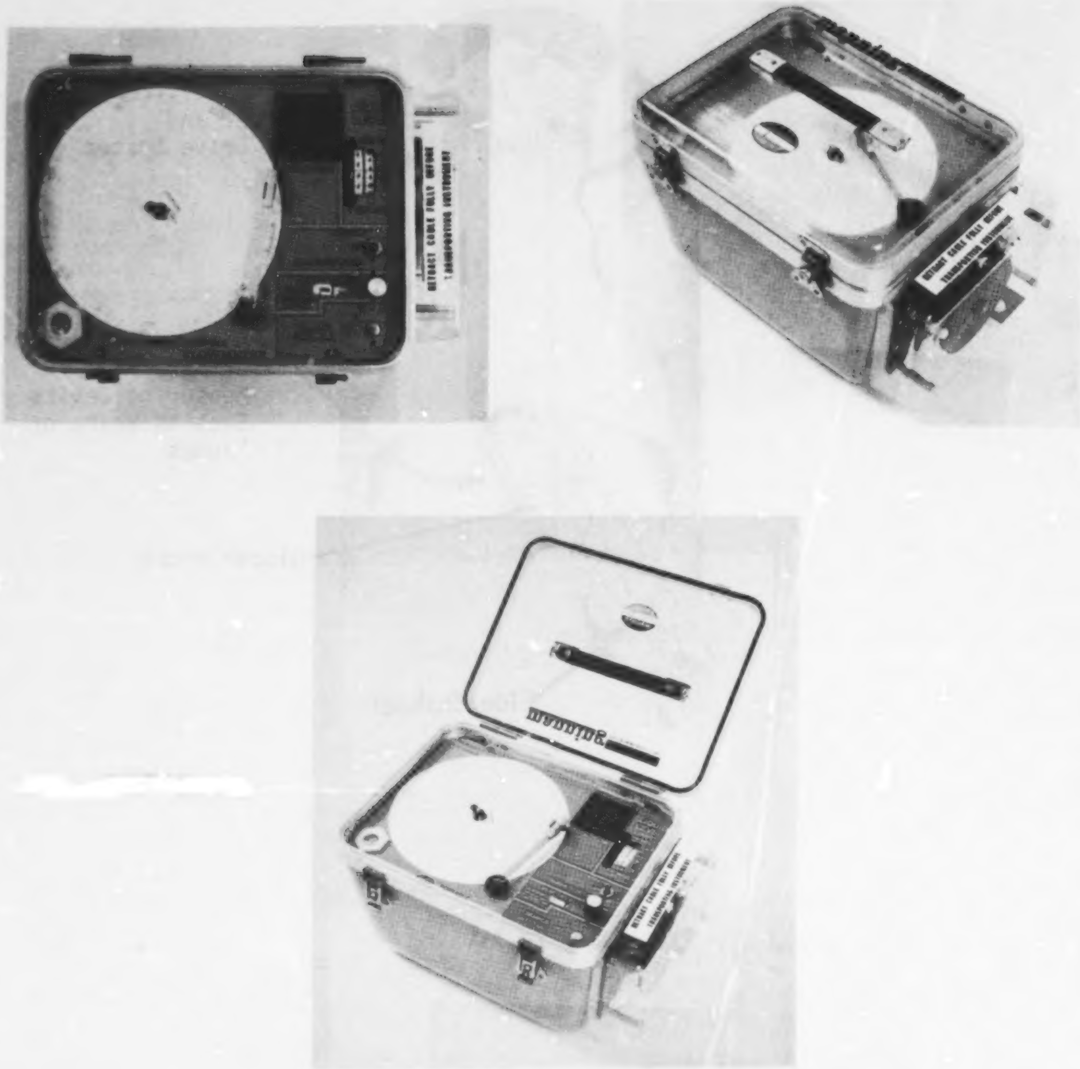

RETRACT CABLE FULLY BEFORE TRANSPORTING INSTRUMENT
manning

FIGURE 20

THE MANNING DIPPER™

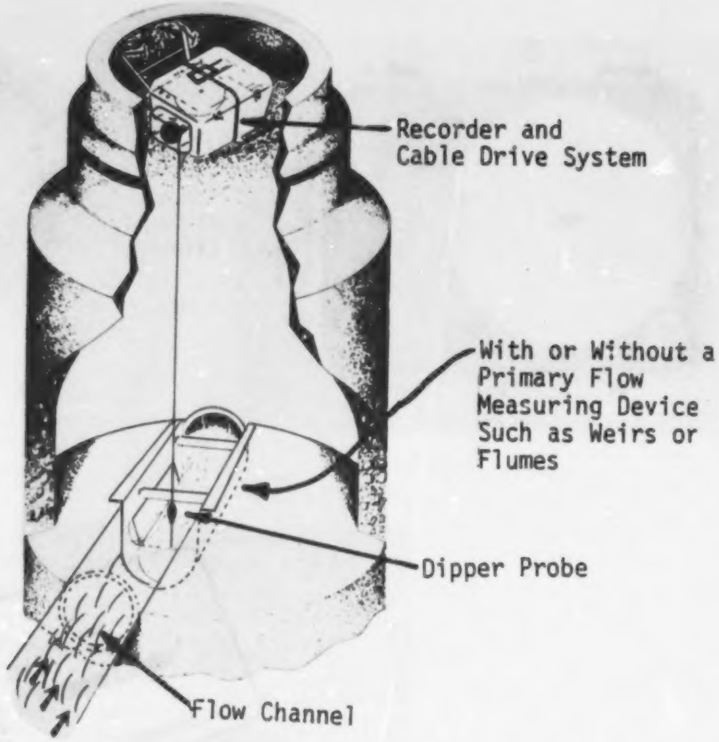

Recorder and Cable Drive System
With or Without a Primary Flow Measuring Device Such as Weirs or Flumes
Dipper Probe
Flow Channel

FIGURE 21

TYPICAL INSTALLATION OF A MANNING "DIPPER"^{TM}
© Manning Environmental Corporation, Santa Cruz. Ca.

On an inspection, the NPDES inspector must look for:

- A bad battery (12 VDC),[20]
- Trash, debris clogging the probe,
- A twisted cable,
- An improperly calibrated Dipper.

The dipper must initially be calibrated by measuring the wastewater level in the sewer by a rule or gauge, or measuring rod.

Belfort Liquid Level Recorders

A Belfort Liquid Level Recorder is shown in Figure 22.* Several EPA Regions use this recorder in their field operations. The advantages and disadvantages of the Belfort Liquid Level Recorder are shown in Table 9.[4]

TABLE 9

FEATURES OF THE BELFORT LIQUID LEVEL RECORDER

ADVANTAGES	DISADVANTAGES
• Rugged • Mechanically simple • Reliable • 6 hr to 8 days/chart revolution • Can be repaired in the field	• Stilling well offers an obstruction to the flow • Hard to install • Almost impossible to install in manholes

---

*The Belfort Instrument Company, Baltimore, Maryland

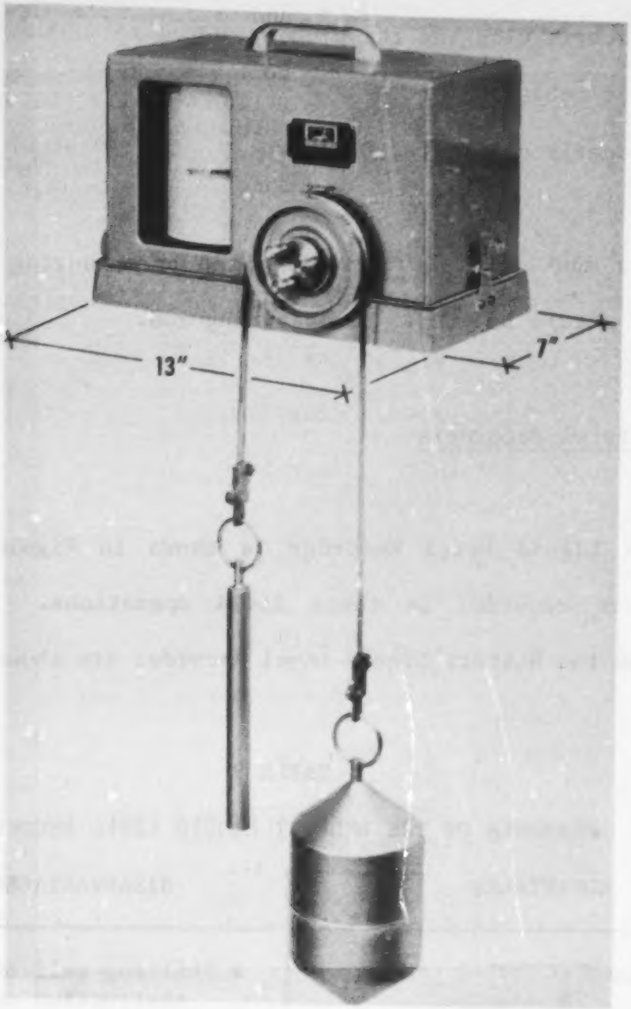


FIGURE 22

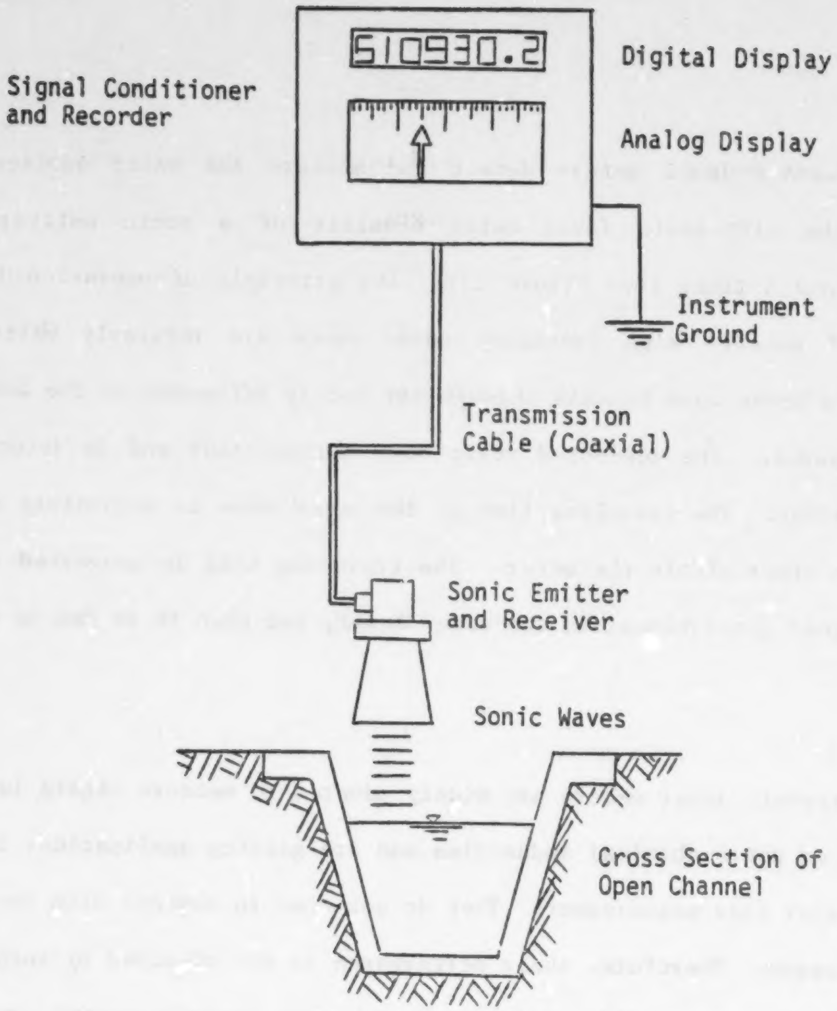
BELFORT LIQUID LEVEL RECORDER

Sonics

Ultrasonic level meters detect and measure the water surface by sonic waves. The ultrasonic level meter consists of a sonic emitter, a sonic receiver, and a timer (see Figure 23). Its principle of operation is the same as that of sonar. High frequency sound waves are regularly emitted by the meter. The sonic wave travels through air and is reflected by the water surface to be measured. The reflected sonic wave bounces back and is detected by the sonic receiver. The traveling time of the sound wave is accurately measured by a built-in timer within the meter. The traveling time is converted to an electrical signal proportional to the water level, and then it is fed to a recorder.

Ultrasonic level meters are widely adopted to measure liquid levels in the petroleum or petro-chemical industries and are gaining applications in the field of wastewater flow measurement. They do not come in contact with the wastewater being measured. Therefore, their performance is not affected by suspended solid loads. They are compatible with almost all open channels except gravity sewers in a manhole. Disadvantages associated with an ultrasonic level meter include:

- inaccurate readings when foams, scums or ice are present in the channel.
- temperature or humidity variations may cause inaccuracies in the readings (some sonic level meters have a temperature compensating device to alleviate these problems to some extent.)



510930.2
Digital Display
Signal Conditioner
and Recorder
Analog Display
Instrument
Ground
Transmission
Cable (Coaxial)
Sonic Emitter
and Receiver
Sonic Waves
Cross Section of
Open Channel

© Endress + Hauser, Inc., Greenwood, IN.

FIGURE 23

SYSTEM LAYOUT OF A SONIC WATER LEVEL METER

Gauges

There are many types of gauges used to measure liquid levels, but the most common are the hook gauges and staff gauges (see Figure 24).

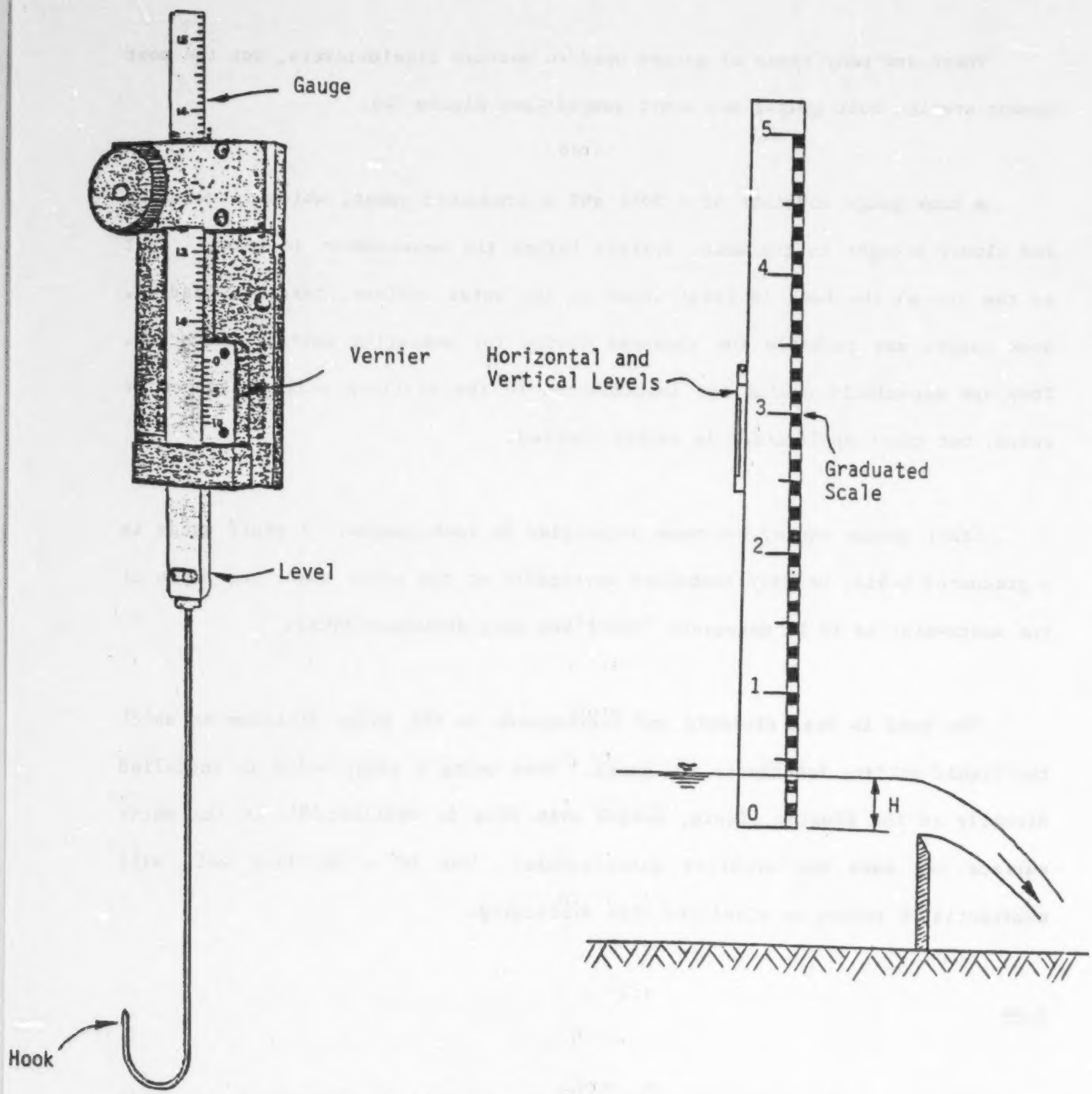
A hook gauge consists of a hook and a graduated gauge, which is manually and slowly brought to the water surface before the measurement is taken. Just as the tip of the hook is first shown on the water surface, take the reading. Hook gauges are probably the cheapest device for measuring surface elevation. They are especially useful for installation in the stilling well of flumes or weirs, but their application is rather limited.

Staff gauges employ the same principles as hook gauges. A staff gauge is a graduated scale, usually installed vertically at the point where the depth of the wastewater is to be measured. There are many different types.

The head is read directly and corresponds to the gauge division at which the liquid surface intersects the gauge. When using a gauge which is installed directly in the flowing liquid, surges with flow or oscillations in the water surface can make the accuracy questionable. Use of a stilling well will substantially reduce or eliminate this difficulty.

Scow

A scow is a floatable block attached to the side of a manhole by a vertically pivoting rod. A wire from the top center of the scow is connected to a


HOOK GAUGE

A TYPICAL STAFF GAGE

Gauge
Vernier
Level
Hook
Horizontal and Vertical Levels
5
4
3
2
1
0
Graduated Scale
H

FIGURE 24

HOOK AND STAFF GAUGES

water level recorder located at the top of the manhole (see Figure 25).

The scow is particularly useful in estimating flows in extremely deep manholes or manholes subject to wide flow fluctuations. Even surcharging conditions can occasionally be monitored by a scow attached to a recorder. However, surcharging conditions can cause the following problems:[19]

(1) Cable slippage,
(2) Recorder destruction,
(3) Unreliability of the recorder charts, and
(4) Ink spillage.

Bubblers

Figure 26 shows a typical installation of a bubbler system for water level measurement. A bubbler system consists of an air supply, a bubbler tube, an air pressure-sensing device and a recorder. The air pressure-sensing device measures the air pressure necessary to push the bubble out of the bubbler tube into the wastewater stream. Usually the bubbler tube is lain as close as possible to the invert of the pipe. The air pressure thus measured by the system is proportional to the depth of the flow.

Bubbler systems are frequently used to monitor the water level in a wet well to control the pumping station which may use multiple pumps or variable speed pumps. Bubbler systems are rarely used for water level measurements in gravity sewers due to their shallower depth of flow as compared to wet well depths.

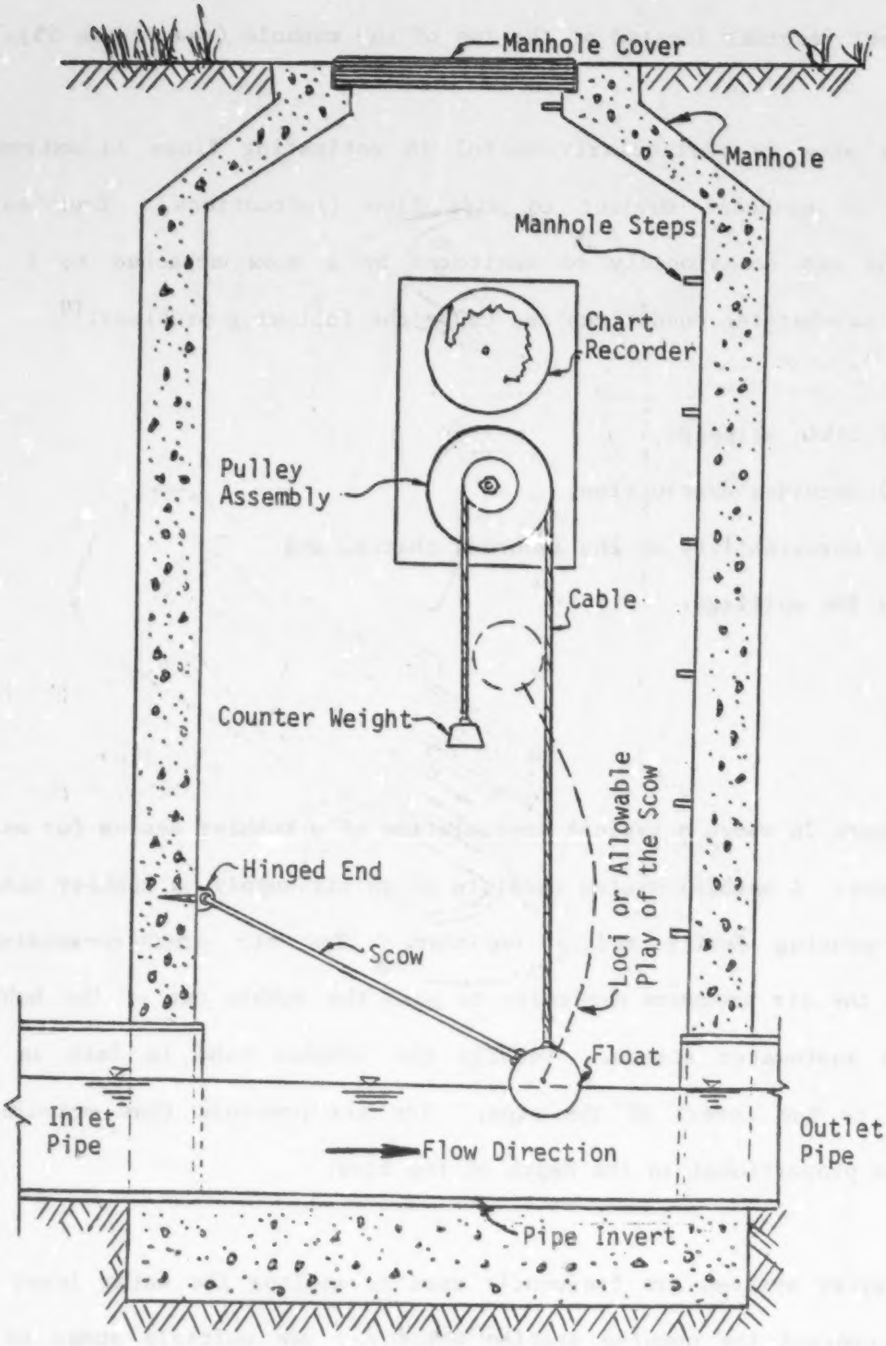

Manhole Cover
Manhole
Manhole Steps
Chart Recorder
Pulley Assembly
Cable
Counter Weight
Loci or Allowable Play of the Scow
Hinged End
Scow
Float
Inlet Pipe
Outlet Pipe
Flow Direction
Pipe Invert

FIGURE 25

TYPICAL INSTALLATION OF A SCOW

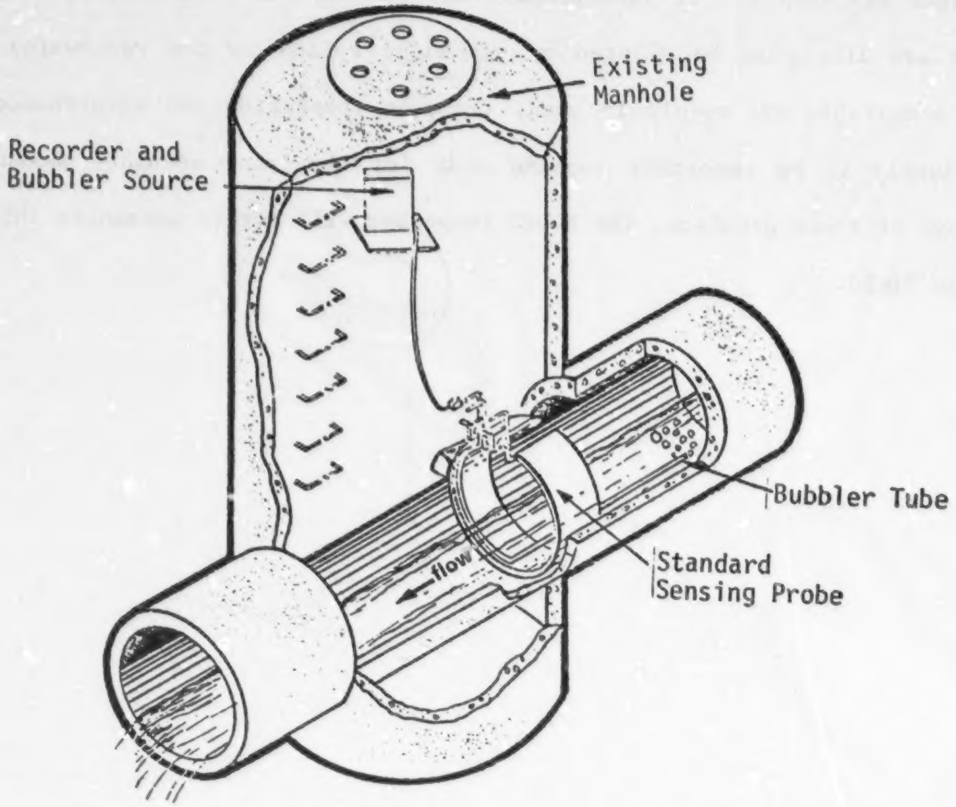

Existing Manhole
Recorder and Bubbler Source
Bubbler Tube
Standard Sensing Probe
flow

FIGURE 26

TYPICAL INSTALLATION OF A BUBBLER

Bubblers can be an accurate method for measuring water surface elevations when they are properly installed and when compatible recorders for the expected range of flow depths are selected. They are relatively expensive and require a reliable air supply. If insufficient air pressure is encountered, the bubbler tubes are likely to be clogged by suspended solids in the wastewater stream. When a portable air supply is used, frequent inspection and maintenance of the air supply is an important routine task for obtaining accurate measurements. Because of these problems, the NPDES inspector will rarely encounter this system in the field.

CHARTS AND CALIBRATIONS

When working with charts be sure to (see Figure 27):

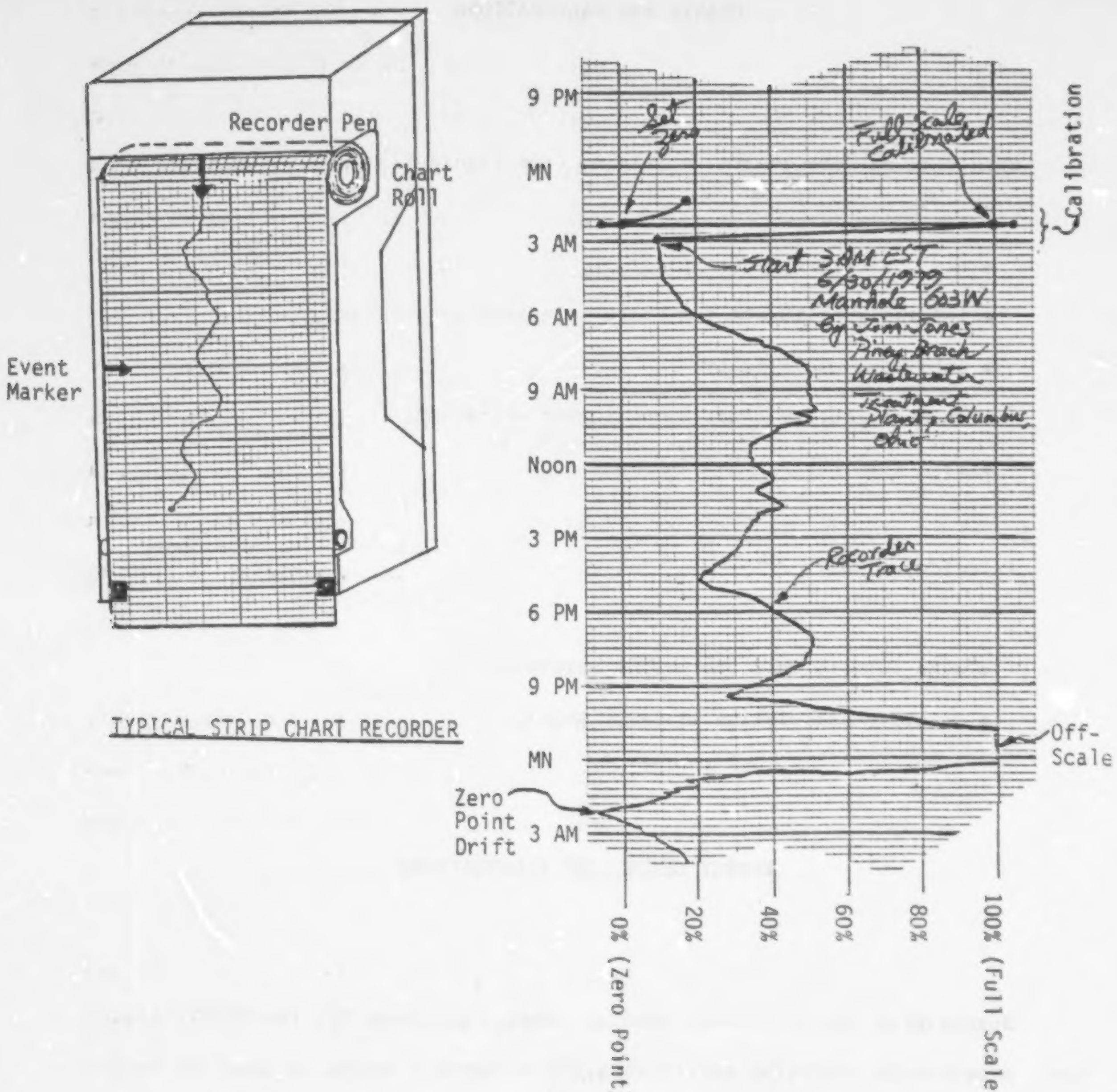
- use the correct chart paper;
- initially calibrate the chart to actual conditions;
- label the chart fully;
- recalibrate the chart when it goes off-scale;
- make sure the pen has ink in it.

In calculations:

- make sure to read the graphs carefully;
- do every calculation at least twice.

ENERGY GRADE LINE CALCULATIONS

Surcharging sewers present special complex problems for the NPDES inspector. A surcharge condition exists when the wastewater height is into the barrel of the manhole above the form groove in the base concrete. Then not only is the sewer flowing full, but there is a certain amount of hydraulic pressure forcing too much wastewater through an inlet not large enough to accomodate the flow, thus increasing the velocity head.


Recorder Pen
Chart Roll
Event Marker
9 PM
MN
3 AM
6 AM
9 AM
Noon
3 PM
6 PM
9 PM
MN
3 AM
Set Zero
Full Scale Calibrated
Calibration
Start 3 AM EST 6/30/1979 Manhole 603W By Jim Jones Piney Branch Wastewater Treatment Plant, Columbus, Ohio
Recorder Trace
Off-Scale
Zero Point Drift
0% (Zero Point)
20%
40%
60%
80%
100% (Full Scale)

TYPICAL STRIP CHART RECORDER

TYPICAL RECORDER CHART

FIGURE 27

TYPICAL STRIP CHART RECORDER AND STRIP CHART

Under these conditions, flows can be calculated occasionally by computing the energy grade line, or the total energy in any section with reference to some elevation or datum, as it is often called. The technique is useful for measuring flows in sewers located directly upstream from pumping stations. The method's reliability, however, is jeopardized by only periodically measuring the flow. A constant flow record is impracticable to obtain by this method.[19]

Theoretical energy grade line calculations accounting for density, pressure, velocity head, pipe friction, etc. are too difficult in the field. To obtain a reasonably accurate flow measurement with simplified calculations, a situation of steady-state hydraulic conditions, no pipe networks nor pipes in series can be assumed. This method cannot be used to calculate flows in siphons or through pumps.

The hydraulic situation is shown in Figure 28 (adapted from reference 10). A key assumption is made that the energy grade line, for all practicable purposes, is parallel to the hydraulic grade line. The datum is taken upstream from the centerline (℄) of the downstream manhole. The method is:

1. Measure the distances Y_1 and Y_3 in feet along ℄ with a plumb line and measuring tape by subtracting the depth from the land surface to the water surface from the total depth from the land surface to the invert.

2. Measure or compute X in feet.

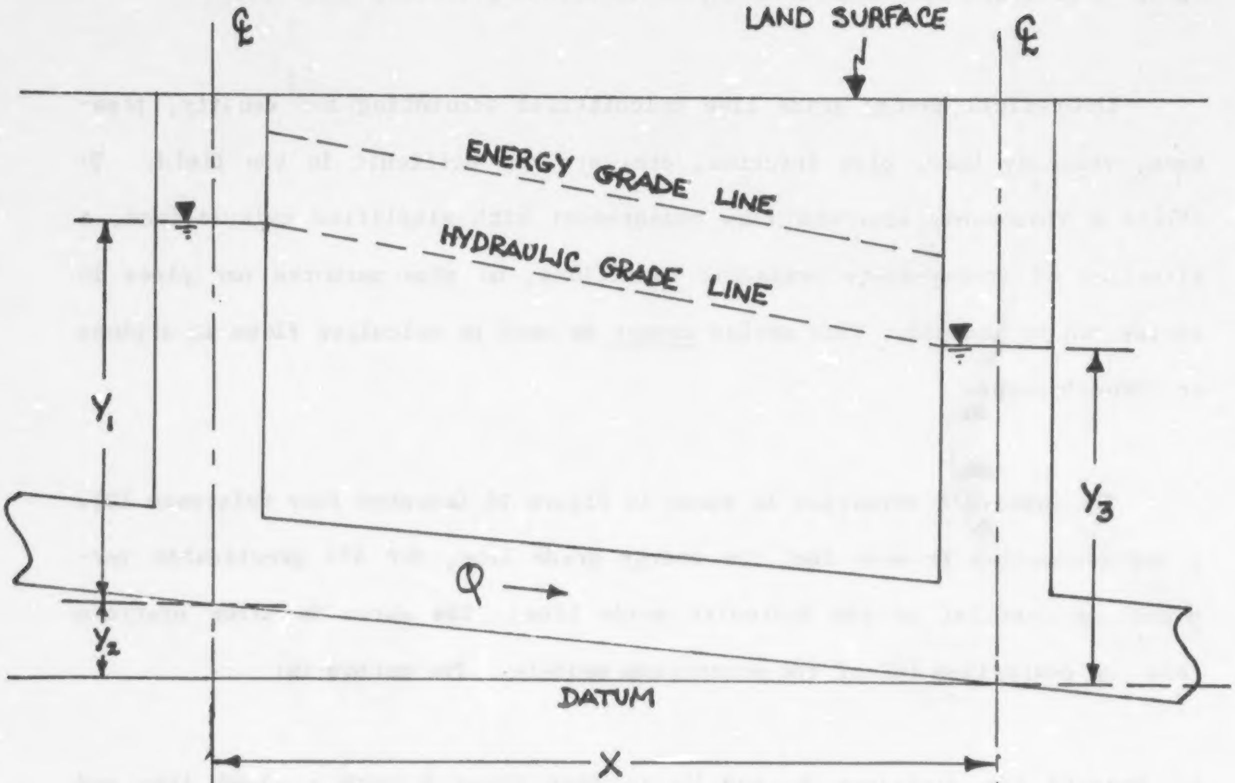

℄
LAND SURFACE
℄
ENERGY GRADE LINE
HYDRAULIC GRADE LINE
Y1
Y2
Y3
Q
DATUM
X

FIGURE 28

SURCHARGING SEWER SCHEMATIC

3. Calculate Y_2 from the slope(s) of the sewer. Example:

   if $X = 350$ ft and $S = 0.001$ ft/ft, then

   $Y_2 = (X)(S) = (350)(0.001) = 0.35$ ft

4. Calculate the slope of the hydraulic grade line (and the energy grade line because they are assumed to be parallel) by:

$$S_e = \frac{Y_1 + Y_2 - Y_3}{X}$$

5. Substitute S_e into the Manning Equation to obtain the velocity (see page 93):

$$V = \frac{1.486}{n} R^{2/3} S_e^{1/2}$$

6. Since the pipe is flowing full, the cross-sectional area of the wastewater is equal to the area of the pipe, so compute the area by simple geometry:

$$A = \pi r^2 = \pi (\frac{D^2}{4})$$

7. Finally, substitute the velocity and area into Darcy's Equation to obtain the flow of the surcharging sewer.

$$Q = VA$$

This algorithm is accurate enough for NPDES use in the field. It can even be applied to infiltration/inflow analysis.

ORIFICES

An orifice is an opening with a closed perimeter of regular form through which water or wastewater flows (if it flows only partially full, then it is a weir, and the standard weir formulas are used to calculate flow).[7] The first work with orifices was done by Torricelli about 1640, and this work lead to the concept of velocity head.[1] See Figure 29 for common orifice shapes and coefficients.[10]

Because the orifice works by velocity head, the velocity must be a function of the force of gravity, or:

$$V = f(g) = f(H)$$

where:

V = velocity

g = acceleration of gravity = 32.2 ft/sec^2

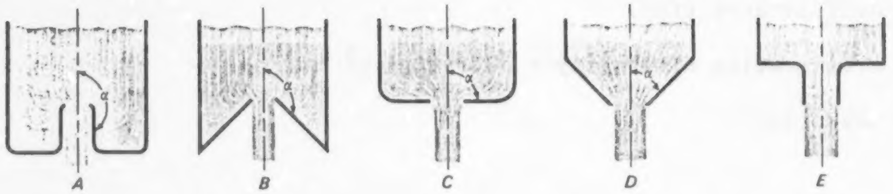
H = velocity head or height of water over the orifice.

because the expression for head loss, h_L, is:

$$h_L = \frac{V^2}{2g}$$

the theoretical velocity (V_t) becomes:

$$V_t = \sqrt{2gH}$$



	A		B		C	D					E
α	180°	157½°	135°	112½°	90°	67½°	45°	22½°	11¼°	5¾°	0°
C	0.541	0.546	0.577	0.606	0.632	0.684	0.753	0.882	0.924	0.949	0.966

FIGURE 29

ORIFICE SHAPES AND THEIR COEFFICIENTS

A constant must be included to relate the theoretical velocity V_t to the actual velocity V. This constant, the coefficient of contraction, describes the opening of the orifice. Substituting the above theoretical velocity equation into Darcy's Law, or Q = VA, and applying C yields the discharge equation for an orifice:

$$Q = Ca\sqrt{2gH}$$

where:

Q = flow (cfs)

C = coefficient of contraction

a = orifice area (ft^2)

g = acceleration of gravity = 32.2 ft/sec^2

H = head (ft)

Example: A 75-mm-diameter orifice type "B" with $\alpha = 112.5°$ has a head of 4.88 inches. What is the flow?

$$Q = Ca\sqrt{2gH}$$

$$a = \pi r^2 = 3.1416 \times \left(\frac{75\text{mm}}{2} \times \frac{0.03937 \text{ in.}}{\text{mm}} \times \frac{1 \text{ ft.}}{12 \text{ in.}}\right)^2$$

$$\sqrt{2gH} = \left[2(32.2)\left(4.88 \text{ in.} \times \frac{1 \text{ ft.}}{12 \text{ in.}}\right)\right]^{1/2}$$

$$C = 0.606 \text{ (From Figure 29)}$$

$$Q = (0.606)(0.387)(26.19)^{1/2} = 1.20 \text{ cfs}$$

Note: Convert all values to proper units.

NOZZLES

Experimentation with nozzles started in 1888 by John R. Freeman, who applied his studies to fire hoses.[16] The general equation for a nozzle is almost identical to that for an orifice.[18]

$$Q = Ca\sqrt{2gh}$$

where:

a = area of the nozzle, ft^2
C = constant for the nozzle, varying per the type of nozzle
g = force of gravity = 32.2 ft/sec^2
h = head, ft

The flow nozzle is cheaper than a venturi meter, and it works quite well in wastewaters with high suspended solid loads. A schematic of a flow circular nozzle mounted inbetween the flanges of a pipe is shown in Figure 30. Notice the high pressure and low pressure tap. A manometer is usually connected inbetween these taps.

If a manometer is connected inbetween the taps, a manometer-derivation of the nozzle equation must be used to calculate flow. It is:

$$Q = CA_2\sqrt{2gR'\left(\frac{S_o}{S_1} - 1\right)}$$

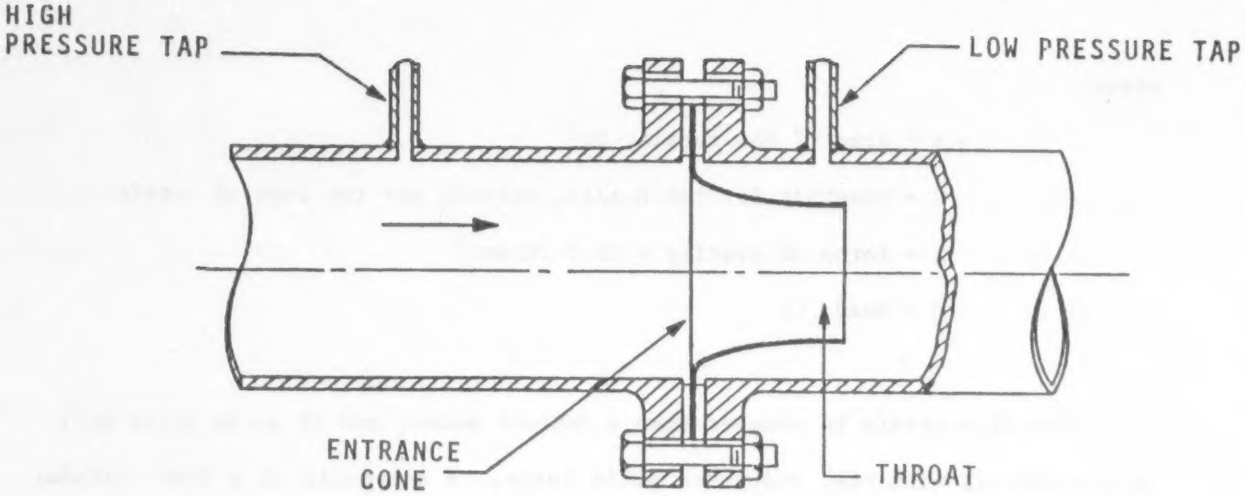

HIGH PRESSURE TAP
LOW PRESSURE TAP
ENTRANCE CONE
THROAT

FIGURE 30

FLOW NOZZLE IN PIPE

(SIDE VIEW)

where:

Q = flow (cfs)

C = nozzle constant

A_2 = area at the downstream end of the nozzle (ft^2)

g = acceleration of gravity = 32.2 ft/sec^2

R' = gage difference (ft)

S_o = specific gravity of the manometer liquid

S_1 = specific gravity of the flowing fluid (e.g., water).

Example:[18] Determine the flow through a 6-in-diameter waterline that contains a 4-in-diameter circular flow nozzle. The mercury-water differential manometer has a gage difference of 10.0 in. at standard conditions (Author's Note: for NPDES purposes, standard conditions can usually be assumed). The nozzle coefficient is 1.056. What is the flow?

$$Q = CA_2\sqrt{2gR'\left(\frac{S_o}{S_1} - 1\right)}$$

From the data given:

$S_o = 13.6$

$S_1 = 1.0$

$R' = 10 \text{ in} \times \frac{1 \text{ ft}}{12 \text{ in}} = 0.833 \text{ ft.}$

$A_2 = \pi\left(\frac{d}{2}\right)^2 = \pi\left(\frac{4 \text{ in} \div 12 \text{ in/ft}}{2}\right)^2 = 0.0873 \text{ ft.}^2$

$g = 32.2 \text{ ft/sec}^2$

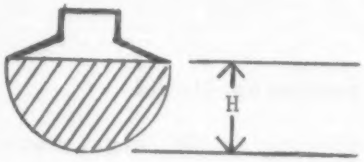
$C = 1.056$

Substituting:

$$Q = (1.056)(0.0873)\left[2(32.2)(0.833)\left(\frac{13.6}{1.0} - 1\right)\right]^{1/2}$$

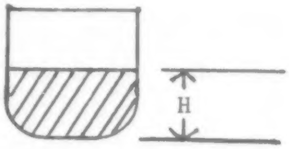
$$Q = 2.40 \text{ cfs}$$

There are two other kinds of nozzles, the Kennison* and the Parabolic. The Kennison nozzle looks like (also see Figure 31):



Its flow is directly proportional to the head, or $Q \propto H$. It is used in pipe diameters from 6 in. to 36 in. and for flows from 0 to 18 mgd (0 to 27.8 cfs). The flow can be calculated by means of the rating curve in Figure 31, or by one supplied by the manufacturer.

The Parabolic nozzle looks like:

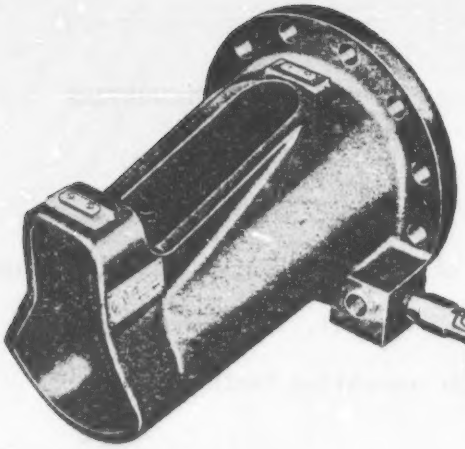


The flow is proportional to the square of the head, or $Q \propto H^2$. The discharge should be read from rating curves supplied by the manufacturer.

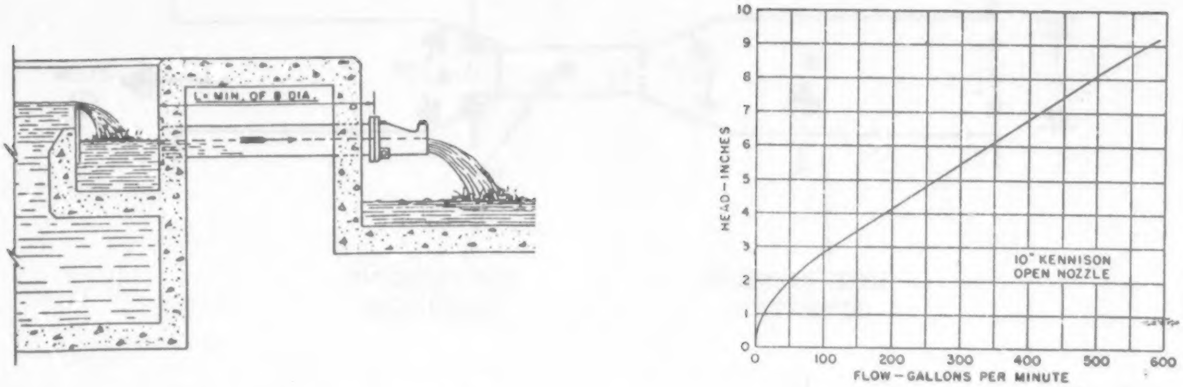
The nozzle should be mounted at the end of a long, straight, horizontal pipe to provide the greatest accuracy. Ten diameters of straight pipe preceding the nozzle is recommended.[18]

---

* The BIF Company



Kennison Nozzle



Installation for Chamber
or Basin Flow

Typical Rating Curve - 10 Inch
Kennison Nozzle (Note Linear Relationship
from 1/10 to Maximum Flow).

FIGURE 31

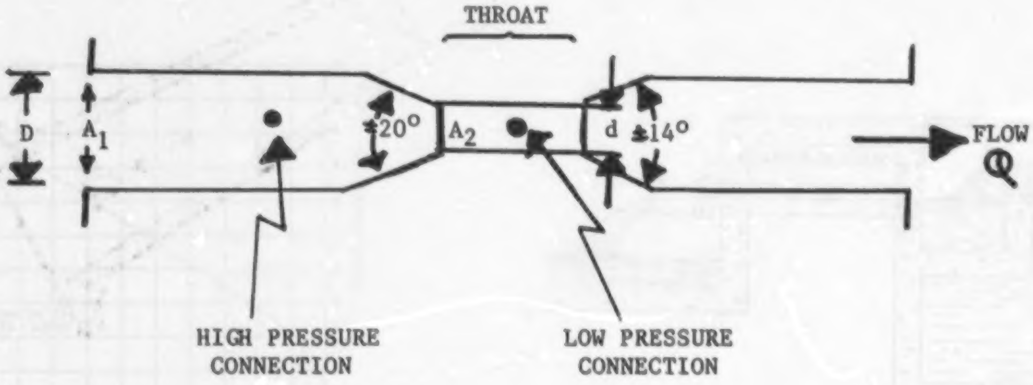
KENNISON OPEN FLOW NOZZLE

PICTORIAL, INSTALLATION, AND TYPICAL RATING CURVE

VENTURI FLOWMETERS

The principle of this apparatus was discovered in about 1791 by the Italian engineer, J.B. Venturi. The principle was first applied in 1887 by Clemens Herschel in the so-called Venturi meter.[10]

A side view of a Venturi meter is shown in Figure 32.[21] However, most Venturis are mounted horizontally in wastewater flow measurement applications, so another drawing appears below:


THROAT
D
A_1
±20°
A_2
d
±14°
FLOW
Q
HIGH PRESSURE CONNECTION
LOW PRESSURE CONNECTION

The Venturi formula is:[10]

$$Q = (1.00 \pm 0.02)\, A_2 \sqrt{2gH}$$

where:

Q = flow, cfs

A_2 = area at the throat, ft^2

g = acceleration of gravity = 32.2 ft/sec^2

H = head, ft = $h_1 - h_2$ (the pressure heads, ft)

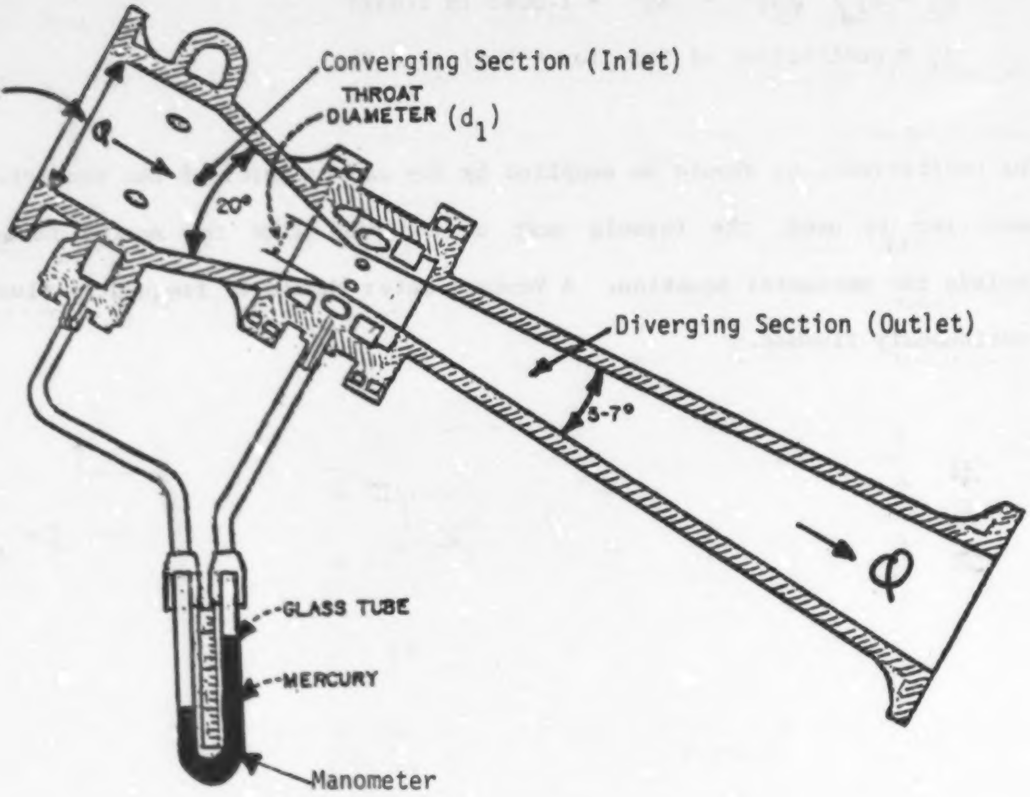

Converging Section (Inlet)
PIPE DIAMETER (d_2)
THROAT DIAMETER (d_1)
Q
20°
Diverging Section (Outlet)
5-7°
Q
GLASS TUBE
MERCURY
Manometer

FIGURE 32

VENTURI METER

(SIDE VIEW)

The coefficient (1.00 ± 0.02) has two parts:

$$c = c_1 c_2 = (1.00 \pm 0.02)$$

$$c_1 = A_1 \Big/ \sqrt{A_1^2 - A_2^2} = 1.0062 \text{ to } 1.0328$$

$$c_2 = \text{coefficient of friction} = 0.97 \text{ to } 0.99$$

The coefficient, c, should be supplied by the manufacturer of the Venturi. If a manometer is used, the formula must be modified like the nozzle formula to include the manometer equation. A Venturi meter should be frequently flushed or continuously flushed.[3]

OPEN-PIPE METHODS

If there is no flow measurement system and the outfall discharges unsubmerged above a receiving water, an open-pipe method can be used to determine flow. There are two main open-pipe methods: The California Pipe Method and the Purdue Method. A third formula is used for approximating flow from vertical pipes.

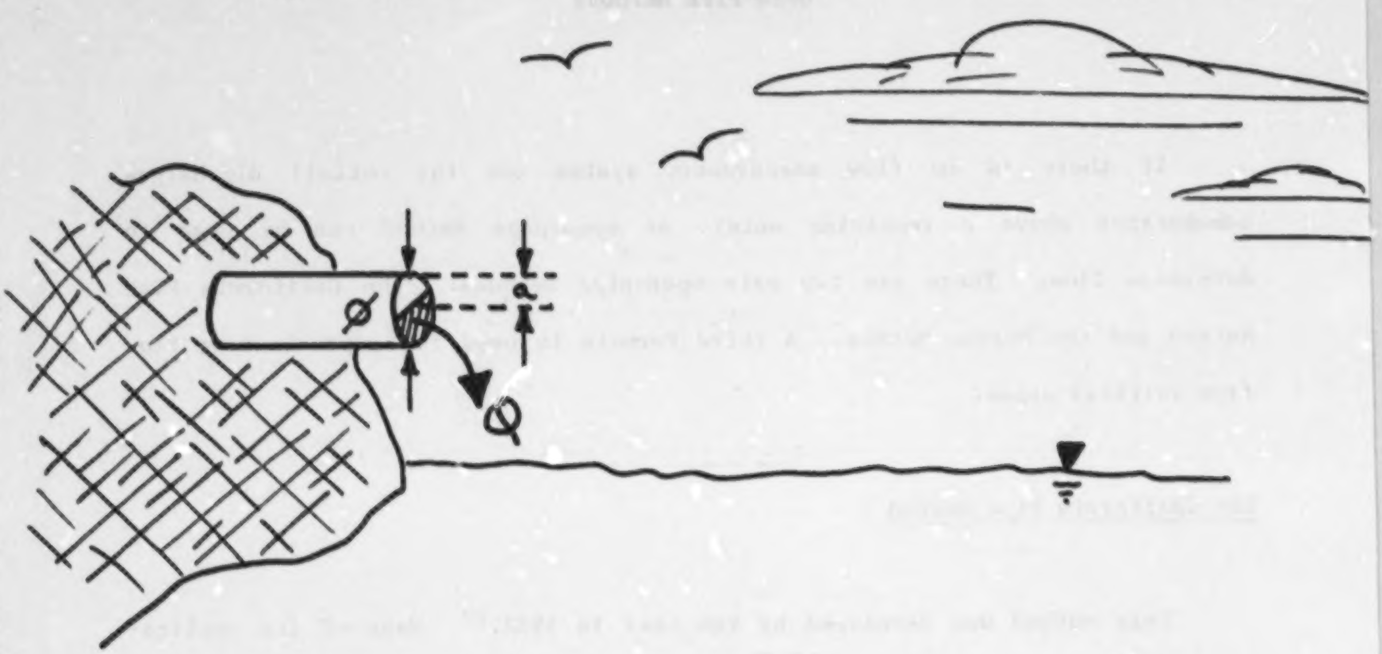
The California Pipe Method

This method was developed by Van Leer in 1922.[22] Many of its applications undoubtably have been for coastal discharges. Four basic criteria must be satisifed for the method to be valid:

(1) The pipe must be level.
(2) The pipe must discharge partially full.
(3) The pipe must discharge freely into the air.
(4) The velocity of approach must be practically zero.

The equation is based on experimental data, and it is good for pipe diameters from 3 to 10 inches (0.25 to 0.83 ft.). It cannot be used with corregated metal pipes.

A support diagram for the California Pipe Method is (see also Figure 33):



The formula for Figure 33, the above drawing, and the California Pipe Method is:

$$Q = 8.69 \left(1 - \frac{a}{\varnothing}\right)^{1.88} \varnothing^{2.48}$$

where:

Q = discharge, cfs

a = distance from the inside crown (top) of the pipe to the water surface measured at the point at which the wastewater discharges from the pipe, ft.

Ø = diameter of the pipe, ft.

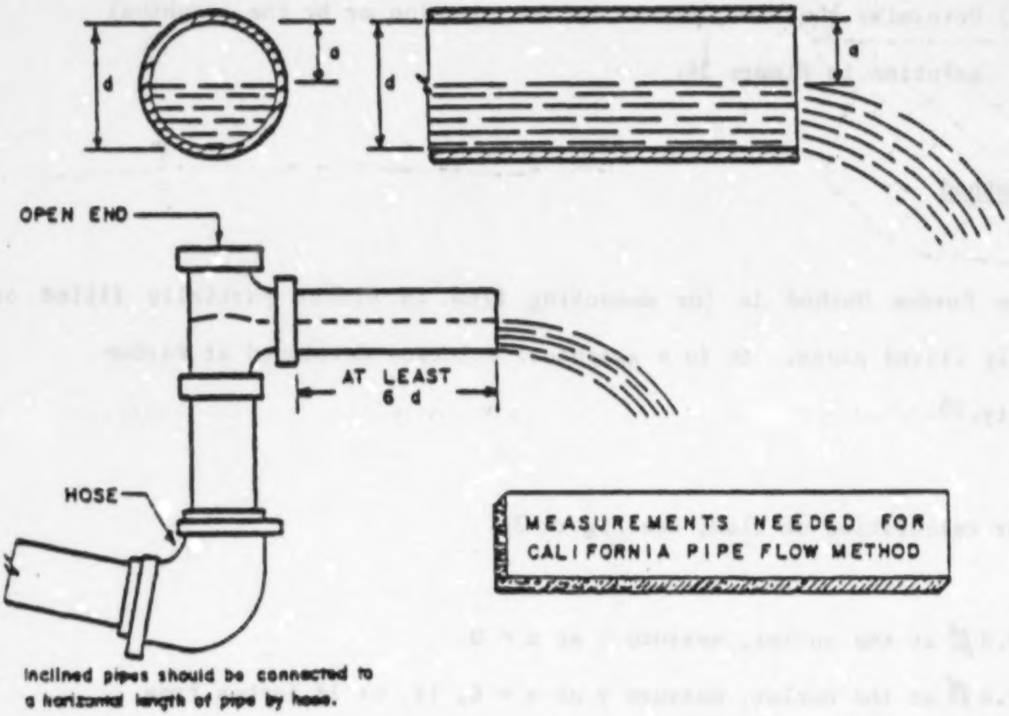

d
a
d
a
OPEN END
AT LEAST 6 d
HOSE
MEASUREMENTS NEEDED FOR CALIFORNIA PIPE FLOW METHOD
Inclined pipes should be connected to a horizontal length of pipe by hose.

FIGURE 33

CALIFORNIA PIPE FLOW METHOD

If $a/\varnothing < 0.5$, the formula does not work.

For solution, the NPDES inspector must:

(1) See if the four criteria are met.

(2) Measure a and $\varnothing$, or obtain $\varnothing$ from the permittee's records.

(3) Determine the flow, either by the equation or by the graphical solution in Figure 34.

Purdue Method

The Purdue Method is for measuring flow in either partially filled or completely filled pipes. It is a graphical solution developed at Purdue University.[23]

For calculation of flow, see Figure 35.

If $d < 0.8\varnothing$ at the outlet, measure y at $x = 0$.

If $d \geq 0.8\varnothing$ at the outlet, measure y at $x = 6$, 12, or 18 inches from the end of the pipe.

Always be sure to use the correct graph! In the above two conditions, d = depth, ft. and $\varnothing = d_{max}$ or the inside diameter of the pipe.[9]

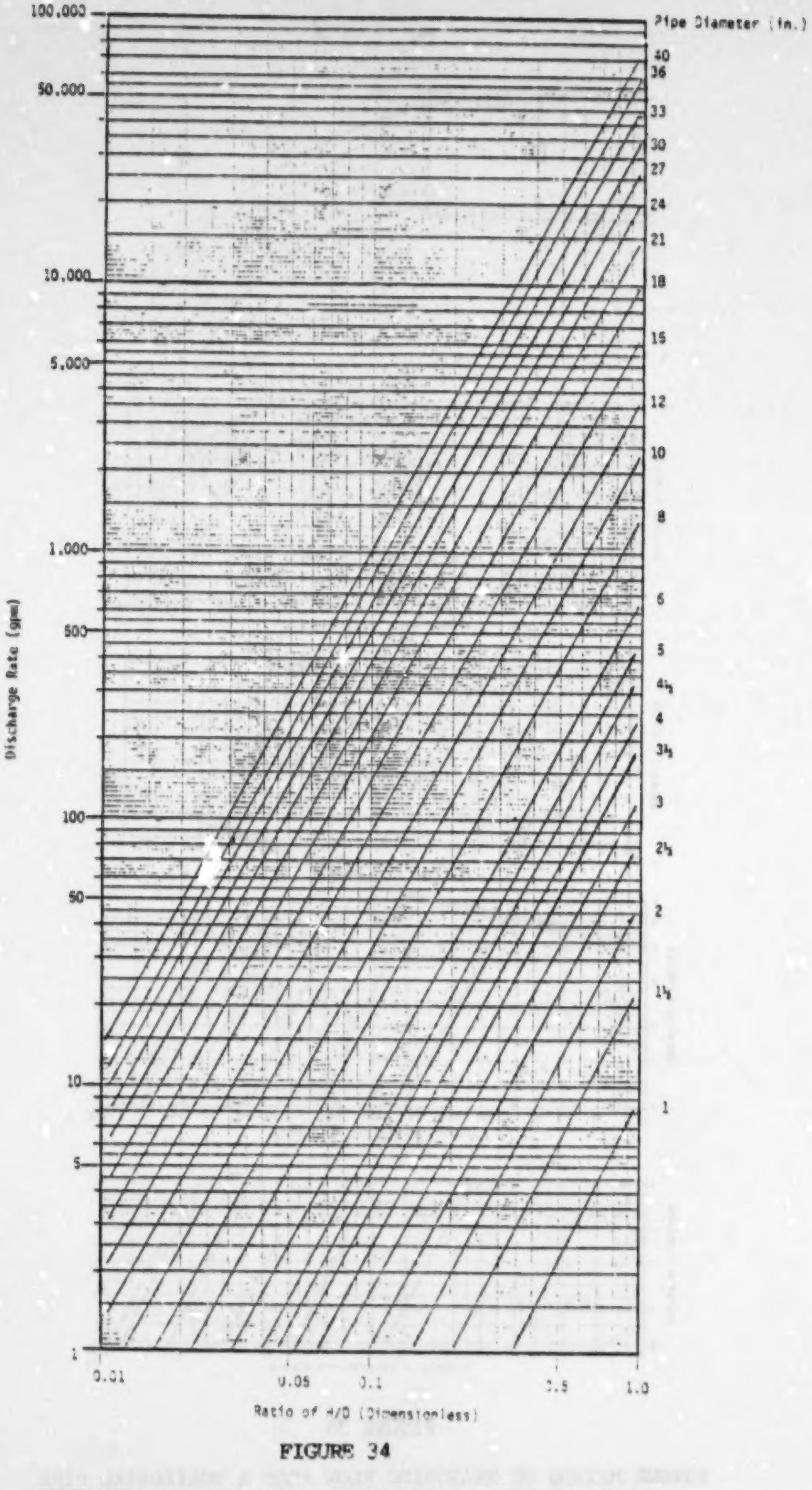


FIGURE 34

DISCHARGE RATE VS. FLOW DEPTH FOR CALIFORNIA PIPES

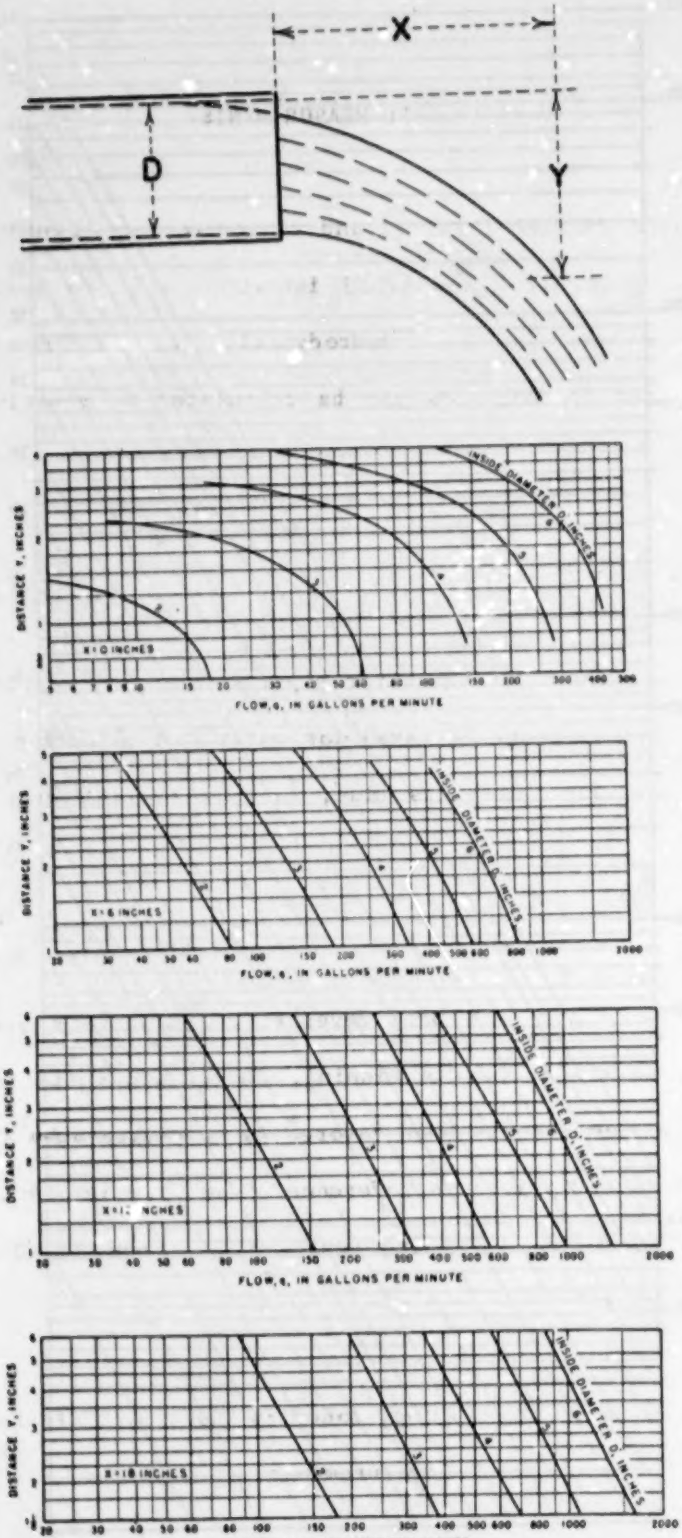


FIGURE 35

PURDUE METHOD OF MEASURING FLOW FROM A HORIZONTAL PIPE

OPEN CHANNEL MEASUREMENTS

So far, open channel (sewer), and closed-conduit flow measurements have been discussed. But often the NPDES inspector will be required to measure streamflow, particularly for hydrodynamic or mathematical modelling applications. Open channel flow can be calculated by equation or by various field methods.

Flow From Vertical Pipes

The NPDES inspector may encounter pipes discharging vertically, especially if flow measurements need to be taken for water and injection wells, aerators, or groundwater supplies. In this case, use the method shown in Figure 36.[29]

Equations

Different equations have been developed to calculate open channel flow. The most famous of these are the Manning, Chezy, and Kutter equations.[7] For years, the Kutter equation has been favored to calculate open channel flow (see solution nomographs and graphs in Reference 8 for more information), but in the last twenty-five years, the Manning equation has been more commonly used in hydraulic engineering.

The Irish engineer, R. Manning, found in 1890, that the Kutter formula did not agree well with many field measurements. He adopted a more accurate and simpler formula:

$$V = \frac{1.486}{n} r^{2/3} S^{1/2}$$

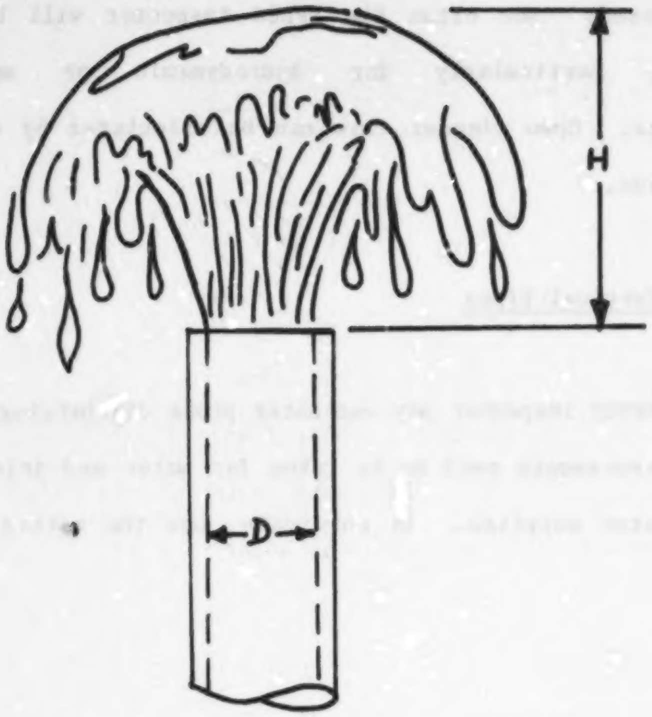


FIGURE 36

APPROXIMATING FLOW FROM VERTICAL PIPE

$$Q = (5.68)\ KD^2\ H^{1/2}$$

where: Q = capacity, gpm
D = inside diameter of pipe, in.
H = vertical height of water jets, in.
K = a constant, varying from 0.87 to 0.97 for pipes 2 to 6 in. diameter and H = 6 to 24 in.

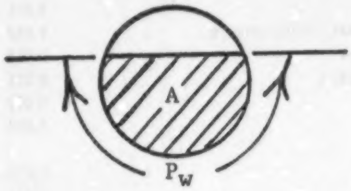
where:

V = velocity, ft/sec or fps

n = roughness coefficient (See Table 10), dimensionless

s = slope, ft/100 ft

r = hydraulic radius = $\frac{\text{area (A)}}{\text{wetted perimeter } (P_w)}$



The term, r, is very difficult to solve for mathematically, but its solution is easy and fast graphically. Graphical solutions for Manning's Equation appear as Figures 37[10] and 38. A nomograph solution is shown in Figure 39.[25] Also, see Reference 8 for more graphical solutions to Manning calculations.

Example: Calculate the flow in a 6 in. concrete pipe in good condition, with a slope of 0.01 and a depth of water equal to 4 in.

From Darcy's Law:

$$Q_{full} = V_f A_{total} \quad \text{or} \quad Q_f = V_f A_T$$

$$A_T = \pi r^2 = \pi\left(\frac{d}{2}\right)^2 = \pi\left(\frac{6 \times \frac{1\text{ ft.}}{12\text{ in.}}}{2}\right)^2$$

$$A_T = 0.196\ \text{ft}^2$$

$$\text{Depth Flow Ratio} = \frac{d}{D} = \frac{4}{6} = 0.67$$

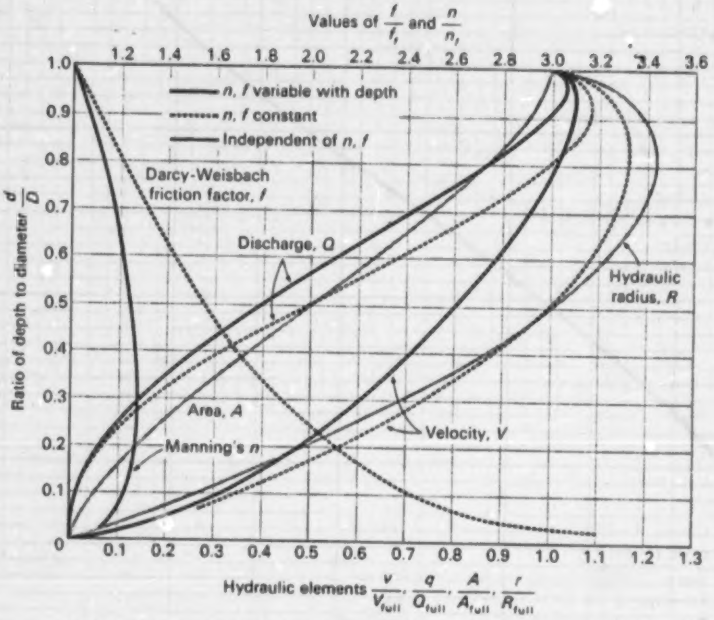
From Figure 38, Enter 0.67 on Y-Axis and read on X-Axis, 0.75 = a/A and from Figure 37, $r/R_{full} = 1.2$.

VALUES OF n TO BE USED WITH THE MANNING EQUATION [7]

Surface	Best	Good	Fair	Bad
Uncoated cast-iron pipe	0.012	0.013	0.014	0.015
Coated cast-iron pipe	0.011	0.012*	0.013*	
Commercial wrought-iron pipe, black	0.012	0.013	0.014	0.015
Commercial wrought-iron pipe, galvanized	0.013	0.014	0.015	0.017
Smooth brass and glass pipe	0.009	0.010	0.011	0.013
Smooth lockbar and welded "OD" pipe	0.010	0.011*	0.013*	
Riveted and spiral steel pipe	0.013	0.015*	0.017*	
Vitrified sewer pipe	0.010 0.011	0.013*	0.015	0.017
Common clay drainage tile	0.011	0.012*	0.014*	0.017
Glazed brickwork	0.011	0.012	0.013*	0.015
Brick in cement mortar; brick sewers	0.012	0.013	0.015*	0.017
Neat cement surfaces	0.010	0.011	0.012	0.013
Cement mortar surfaces	0.011	0.012	0.013*	0.015
Concrete pipe	0.012	0.013	0.015*	0.016
Wood stave pipe	0.010	0.011	0.012	0.013
Plank flumes:				
Planed	0.010	0.012*	0.013	0.014
Unplaned	0.011	0.013*	0.014	0.015
With battens	0.012	0.015*	0.016	
Concrete-lined channels	0.012	0.014*	0.016*	0.018
Cement-rubble surface	0.017	0.020	0.025	0.030
Dry-rubble surface	0.025	0.030	0.033	0.035
Dressed-ashlar surface	0.013	0.014	0.015	0.017
Semicircular metal flumes, smooth	0.011	0.012	0.013	0.015
Semicircular metal flumes, corrugated	0.0225	0.025	0.0275	0.030
Canals and ditches:				
Earth, straight and uniform	0.017	0.020	0.0225*	0.025
Rock cuts, smooth and uniform	0.025	0.030	0.033*	0.035
Rock cuts, jagged and irregular	0.035	0.040	0.045	
Winding sluggish canals	0.0225	0.025*	0.0275	0.030
Dredged earth channels	0.025	0.0275*	0.030	0.033
Canals with rough stony beds, weeds on earth banks	0.025	0.030	0.035*	0.040
Earth bottom, rubble sides	0.028	0.030*	0.033*	0.035
Natural stream channels:				
(1) Clean, straight bank, full stage, no rifts or deep pools	0.025	0.0275	0.030	0.033
(2) Same as (1), but some weeds and stones	0.030	0.033	0.035	0.040
(3) Winding, some pools and shoals, clean	0.033	0.035	0.040	0.045
(4) Same as (3), lower stages, more ineffective slope and sections	0.040	0.045	0.050	0.055
(5) Same as (3), some weeds and stones	0.035	0.040	0.045	0.050
(6) Same as (4), stony sections	0.045	0.050	0.055	0.060
(7) Sluggish river reaches, rather weedy or with very deep pools	0.050	0.060	0.070	0.080
(8) Very weedy reaches	0.075	0.100	0.125	0.150

* Values commonly used in designing.

TABLE 10



Hydraulic elements for circular sewers

FIGURE 37

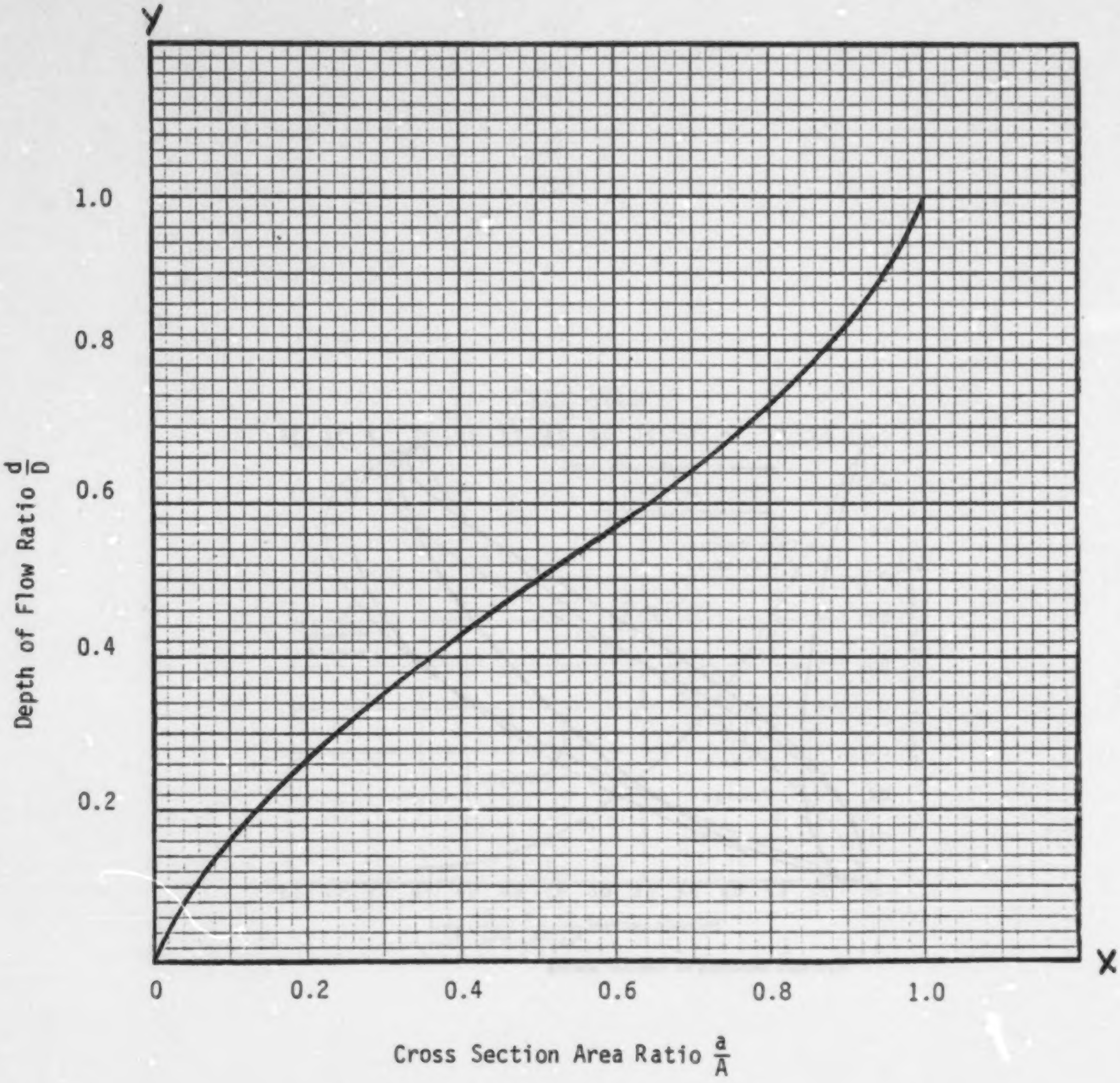


FIGURE 38

DEPTH RATIO VERSUS AREA RATIO

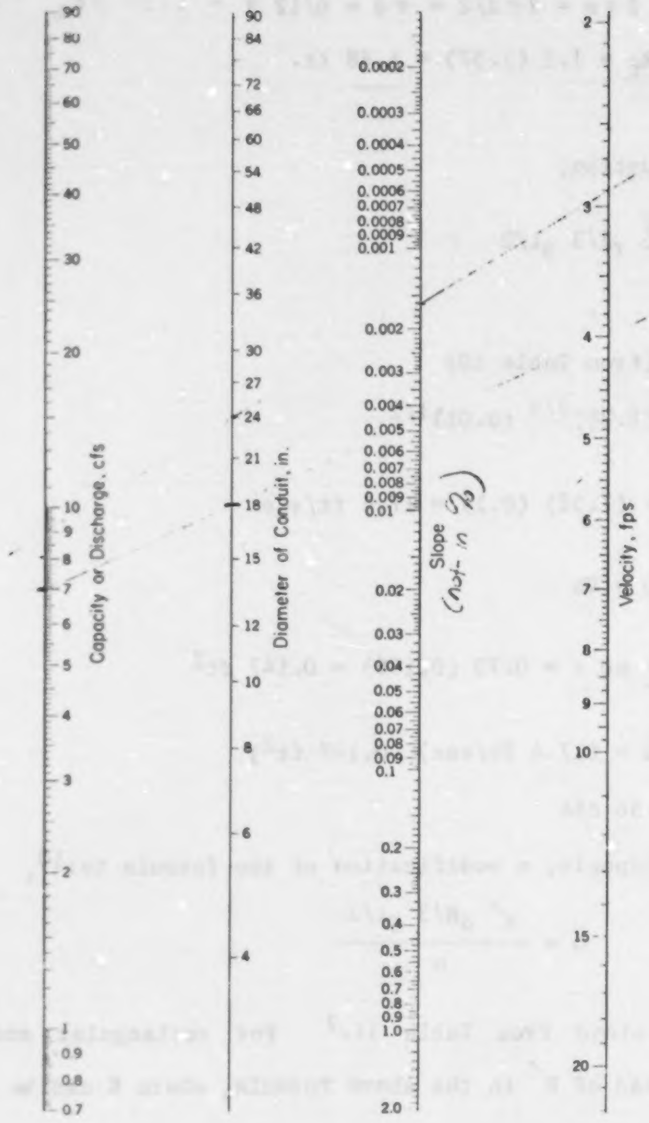


FIGURE 39

NOMOGRAPH BASED ON MANNING's FORMULA, n = 0.013

So, if $R_{full} = 2\pi r = 2\pi d/2 = \pi d = 6/12\ \pi = 1.57$ ft.

Then, $r = 1.2\ R_f = 1.2\ (1.57) = 1.88$ ft.

Using the Manning Equation,

$$V = \frac{1.486}{n} r^{2/3} S^{1/2}$$

where:

$n = 0.013$ (From Table 10)

$$V = \frac{1.486}{0.013} (1.88)^{2/3} (0.01)^{1/2}$$

$$V = (114.3)\ (1.52)\ (0.1) = 17.4 \text{ ft/sec}$$

Using Darcy's Law, $Q = Va$

And, $0.75 = \dfrac{a}{A}$ so $a = 0.75\ (0.196) = 0.147 \text{ ft}^2$

$$Q = Va = (17.4 \text{ ft/sec})\ (0.147 \text{ ft}^2)$$

$$Q = 2.56 \text{ cfs}$$

For circular channels, a modification of the formula is:[10],

$$Q = \frac{K' d^{8/3} S^{1/2}}{n}$$

where K' can be obtained from Table 11.[7] For rectangular and trapezoidal channels, use K instead of K' in the above formula, where K can be obtained also from Table 11.

Velocity-Area Method

Velocity area methods are extremely easy, but rather time consuming methods by which to determine streamflow. All of these methods involve using a current meter to determine the component velocities. Current meters are discussed in the next section on Stream Gauging.

Values of K for Circular Channels in the Formula

$$Q = \frac{K}{n} D^{8/3} s^{1/2}$$

D = depth of water d = diameter of channel

$\frac{D}{d}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0		15.02	10.56	8.57	7.38	6.55	5.95	5.47	5.08	4.76
.1	4.49	4.25	4.04	3.86	3.69	3.54	3.41	3.28	3.17	3.06
.2	2.96	2.87	2.79	2.71	2.63	2.56	2.49	2.42	2.36	2.30
.3	2.25	2.20	2.14	2.09	2.05	2.00	1.96	1.92	1.87	1.84
.4	1.80	1.76	1.72	1.69	1.66	1.62	1.59	1.56	1.53	1.50
.5	1.470	1.442	1.415	1.388	1.362	1.336	1.311	1.286	1.262	1.238
.6	1.215	1.192	1.170	1.148	1.126	1.105	1.084	1.064	1.043	1.023
.7	1.004	.984	.965	.947	.928	.910	.891	.874	.856	.838
.8	.821	.804	.787	.770	.753	.736	.720	.703	.687	.670
.9	.654	.637	.621	.604	.588	.571	.553	.535	.516	.496
1.0	.463									

Values of K' for Circular Channels in the Formula

$$Q = \frac{K'}{n} d^{8/3} s^{1/2}$$

D = depth of water d = diameter of channel

$\frac{D}{d}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0		.00007	.00031	.00074	.00138	.00222	.00328	.00455	.00604	.00773
.1	.00967	.0118	.0142	.0167	.0195	.0225	.0257	.0291	.0327	.0366
.2	.0406	.0448	.0492	.0537	.0585	.0634	.0686	.0738	.0793	.0849
.3	.0907	.0966	.1027	.1089	.1153	.1218	.1284	.1352	.1420	.1490
.4	.1561	.1633	.1705	.1779	.1854	.1929	.2005	.2082	.2160	.2238
.5	.232	.239	.247	.255	.263	.271	.279	.287	.295	.303
.6	.311	.319	.327	.335	.343	.350	.358	.366	.373	.380
.7	.388	.395	.402	.409	.416	.422	.429	.435	.441	.447
.8	.453	.458	.463	.468	.473	.477	.481	.485	.488	.491
.9	.494	.496	.497	.498	.498	.498	.496	.494	.489	.483
1.0	.463									

TABLE 11

For the following discussion, refer to Figure 40. From Figure 40(b) it can be seen that the velocity profile in a stream is not uniform; it varies due to frictional forces found at the sides and bottom of the channel. Thus, several current measurements are needed to accurately determine streamflow.

There are three different ways to determine streamflow. All can be used by the NPDES inspector with relative ease. Other methods are discussed fully in References 9, 24, and 26.

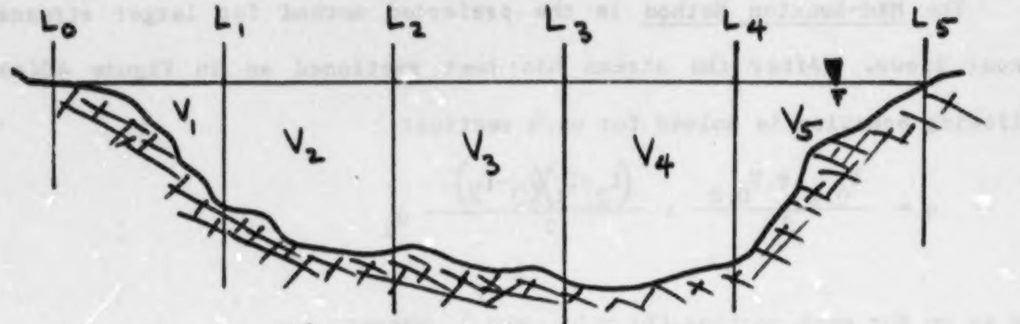
The three methods are the Six-Tenths Depth Method (Figure 40(c)), the Two-Point Method (Figure 40(d)), and the Mid-Section Method. Each involves dividing the stream into equally long (measured from bank to bank in a perpendicular line to streamflow) segments and measuring the flow in each. This is shown in Figure 40(a). The flows per segment are added to obtain the total streamflow.

The Six-Tenths-Depth Method is for shallow flows. It requires a measurement at 0.6d from the water surface. Darcy's Law, $Q = VA$, is used to determine the streamflow.

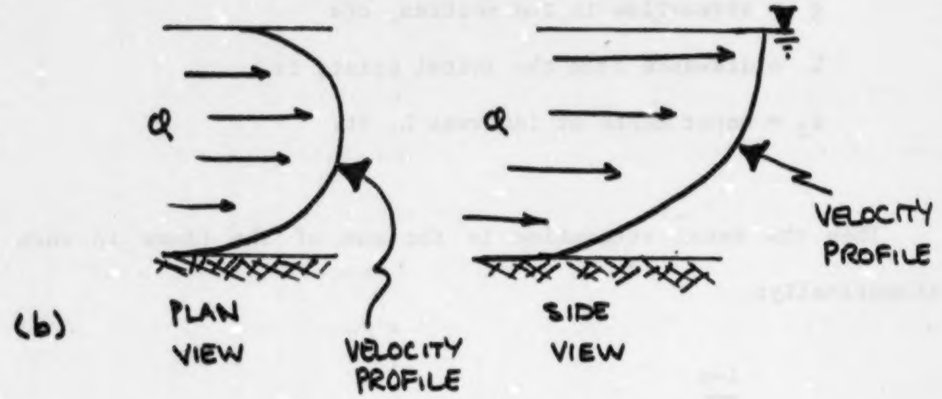
The Two-Point Method is for deeper flows than 2 or 3 feet, and it requires that two readings be taken, at 0.2d and 0.8d from the water surface. The velocities are averaged as follows:

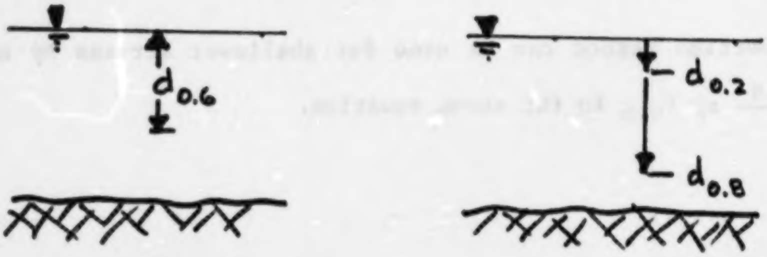
$$V_{avg} = \frac{V_{0.2} + V_{0.8}}{2}$$

Then the streamflow is calculated by Darcy's Law, $Q = V_{avg} A$.


L0
L1
L2
L3
L4
L5
V1
V2
V3
V4
V5

(a) STREAM CROSS-SECTION


Q
Q
VELOCITY PROFILE
VELOCITY PROFILE
(b)
PLAN VIEW
SIDE VIEW



(c) SIX-TENTHS DEPTH METHOD (d) TWO-POINT METHOD

FIGURE 40

DETERMINING MEAN VELOCITIES

The Mid-Section Method is the preferred method for larger streams with deeper flows. After the stream has been sectioned as in Figure 40(a), the following equation is solved for each section:

$$q = \frac{V_{0.2} + V_{0.8}}{2} \times \frac{(L_2 - L_1)(L_3 - L_2)}{2} d_2$$

and so on for each section ($L_4 - L_3$, etc.), where:

q = streamflow in the section, cfs

L = distance from the inital point, ft.

d_2 = water depth at interval L, ft.

Then the total streamflow is the sum of the flows in each section, or mathematically:

$$Q_T = \sum_{n=1}^{1-n} q$$

The Mid-Section Method can be used for shallower streams by substituting $\frac{V_{0.2} + V_{0.8}}{2}$ by $V_{0.6}$ in the above equation.

STREAM GAUGING

Stream gauging has been carried out by various governmental agencies since the mid-nineteenth century.* To measure stream velocity, the NPDES inspector must use several different pieces of equipment. It is best to prepare a checklist before going out in the field.

Essential Equipment is:

- Current meter
- Wading rod (to measure depth and set the current meter to a desired depth).
- Sound box or Earphones (to hear the clicks, one/revolution of the cups or blades on the current meter).
- Stopwatch (to time the number of clicks/minute).
- Clipboard and forms (on which to record data).

---

*See References 9, 24, and 26 for extensive detail on this subject.

This equipment is available from several companies, including:

Weather Measure Corporation
P.O. Box 41257
Sacramento, CA 95841

Kahlsico Scientific
P.O. Box 1166
El Cajon, CA 92022

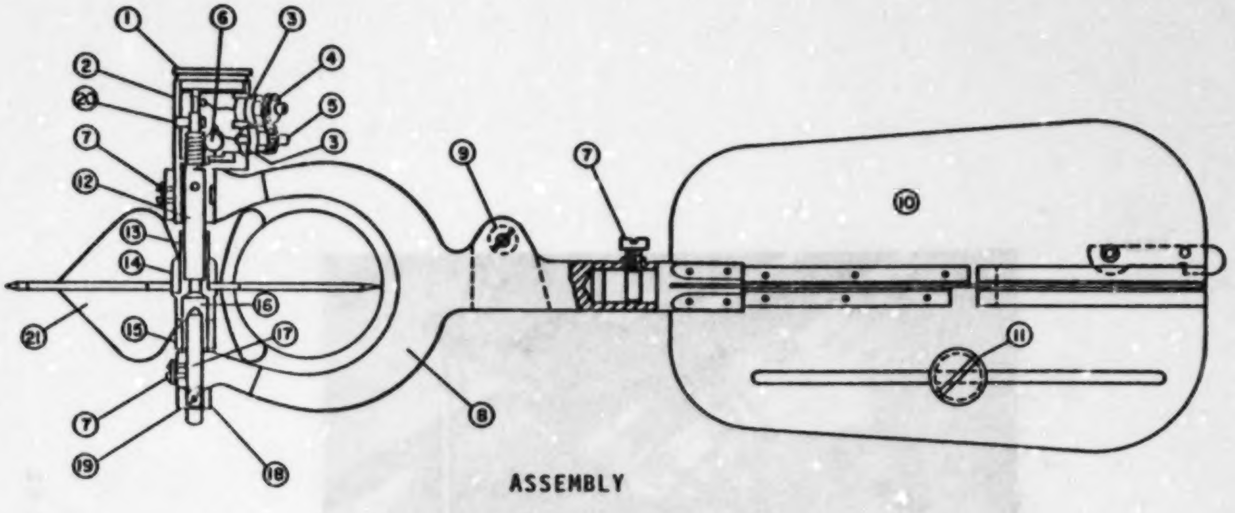
EPIC, Incorporated
150 Nassau Street
New York, NY 10038

For ease of non-smearing and for making duplicate copies of data, rubberized paper forms are available from the Government Printing Office, Washington, D.C.[4] Order by GSA #7-EPA-5300-1.

Current Meters

There are two popular types of current meters used today--the Price and the Ott meters. Others are described in Reference 9.

The Price Meter is used extensively by the U.S. Geological Survey. It is shown schematically in Figure 41 and in action in Figure 42. The Price Current Meter is similar to an anemometer, and each full revolution of one cup causes a



ASSEMBLY

LIST OF PARTS

1. CAP FOR CONTACT CHAMBER
2. CONTACT CHAMBER
3. INSULATING BUSHING FOR CONTACT BINDING POST
4. SINGLE-CONTACT BINDING POST
5. PENTA-CONTACT BINDING POST
6. PENTA GEAR
7. SET SCREWS
8. YOKE
9. HOLE FOR HANGER SCREW
10. TAILPIECE
11. BALANCE WEIGHT
12. SHAFT
13. BUCKET-WHEEL HUB
14. BUCKET-WHEEL HUB NUT
15. RAISING NUT
16. PIVOT BEARING
17. PIVOT
18. PIVOT ADJUSTING NUT
19. KEEPER SCREW FOR PIVOT ADJUSTING NUT
20. BEARING LUG
21. BUCKET WHEEL

FIGURE 41

ASSEMBLY DRAWING OF PRICE TYPE AA CURRENT METER

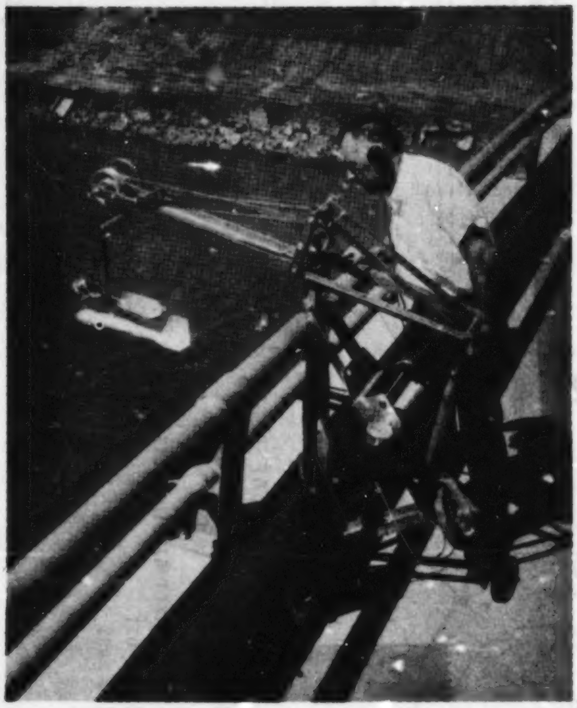


FIGURE 42

TYPE "A" CRANE AND CURRENT METER ASSEMBLY IN POSITION ON A BRIDGE click to be heard in the attached earphones. The pygmy current meter is a smaller version of the Price meter for use on shallow streams.

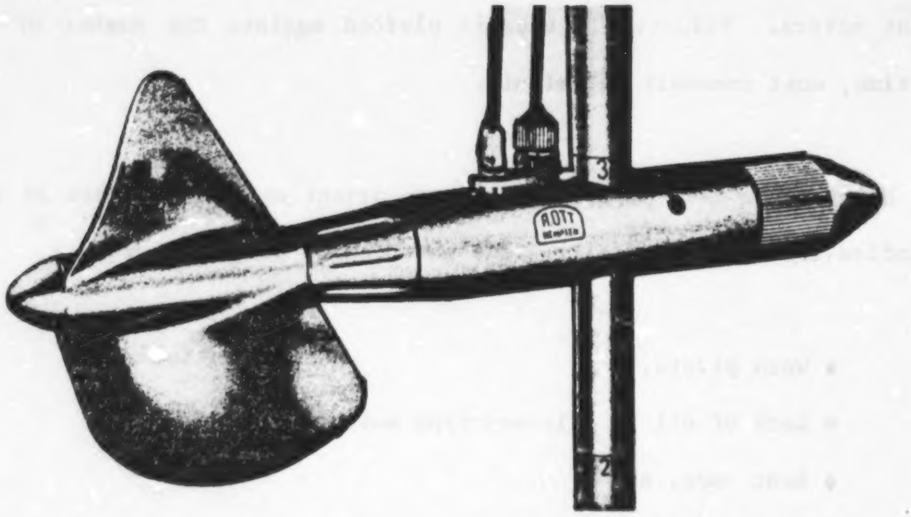
The Ott current meter is one of several propeller-type, horizontal axle meters in use (see Figure 43). Others include Hoff, Haskell, and "Dumas" meters.[9] Ott meters are useful where vertical velocity gradients are a problem.

Flows can be determined by using the rating tables furnished with the current meters. Velocity is usually plotted against the number of clicks per unit time, most commonly per minute.

Maintenance is a problem with most current meters, and lack of it can skew streamflow results drastically. The most common problems are:[4]

- Worn pivots,
- Lack of oil (a silicone-type must be used),
- Bent cups, and
- Solids build-up.

The NPDES inspector must carefully check the current meter for these problems before and after each use. If the meter is used in extremely contaminated or salt water, it must be washed off thoroughly with fresh water before storing.



3
A.OTT
KEMPTEN
2

FIGURE 43

OTT-TYPE HORIZONTAL AXIS CURRENT METER

DILUTION METHODS AND TRACERS

Dilution methods for water and wastewater flow are based on the color, conductivity, or fluorescence of a tracer injected into the waste stream. The advantages and disadvantages of this flow measurement method are:

TABLE 12

COMPARISON OF MERITS OF THE DILUTION METHOD

ADVANTAGES	DISADVANTAGES
• Method which can be used where other methods are inappropriate or impossible.	• Requires special equipment.
• Good for measuring large flows (billions of gallons/day).	• Detailed procedures.
• High accuracy reference procedure to check devices in situ (on site).	• Time-consuming.
• Procedure to verify closed conduit flow measuring systems.	• Costly.

Dilution

There are two methods: a slug-dose can be injected, or the tracer can be injected continuously. There are merits to each method. Tracers used include sodium chloride (NaCl)--sometimes called the salt-velocity method or the salt-dilution method for slug addition[9]--and lithuim chloride (LiCl). Fluorescent dyes, like rhodamine B and Pontacyl Brilliant Pink B, have been extensively used in ocean outfall studies. Submersible pumps with flow-through fluorometers and recorders can also be employed to measure flow by this method.

Slug vs. Constant-Rate Injection

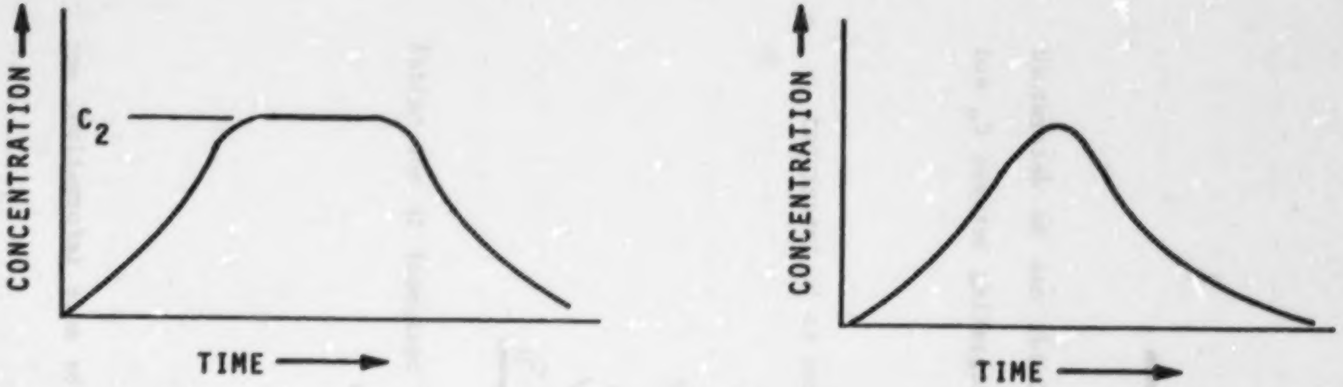
A comparison of the mathematical expressions used to calculate flow and the concentration-time curves for each of the above methods is presented as Figure 44.[27]

For continuous injection, if q, the constant flow rate of the injected tracer, is much smaller than the flow, and if the optimum concentration (of the plateau) is much greater than the background concentration, then the equation can be simplified to:

$$Q = q \frac{C_1}{C_2}$$

But:

- q is a constant injected by perhaps a piston chemical metering pump;
- the tracer must not degrade or sorb onto other particles, and
- the dye must be well-mixed across the section so the following relationship results:



a. CONCENTRATION-TIME CURVE FOR CONSTANT-RATE INJECTION METHOD.

$$Q = \left(\frac{C_1 - C_2}{C_2 - C_0}\right) q$$

Q = IS FLOW RATE OF STREAM

q = IS FLOW RATE OF CHEMICAL

C_0 = IS BACKGROUND CONCENTRATION OF STREAM

C_1 = IS CONCENTRATION OF CHEMICAL INJECTED

C_2 = IS CONCENTRATION OF STREAM PLATEAU

b. CONCENTRATION-TIME CURVE FOR SLUG-INJECTION METHOD.

$$Q = \frac{v\ C_1}{\int_0^\infty (C_2 - C_0)\,dt}$$

Q = IS FLOW RATE OF STREAM

v = IS VOLUME OF CHEMICAL INJECTED

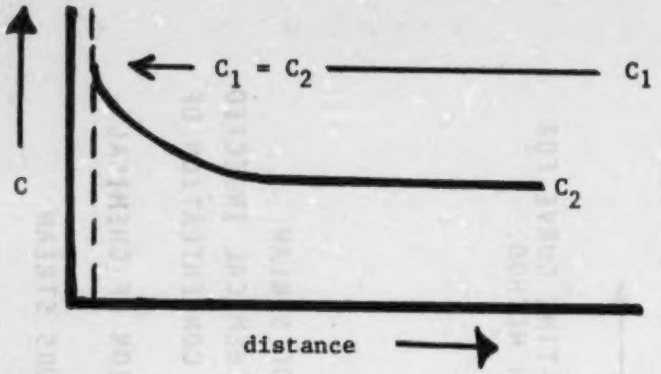
C_0 = IS BACKGROUND CONCENTRATION OF STREAM

C_1 = IS CONCENTRATION OF CHEMICAL INJECTED

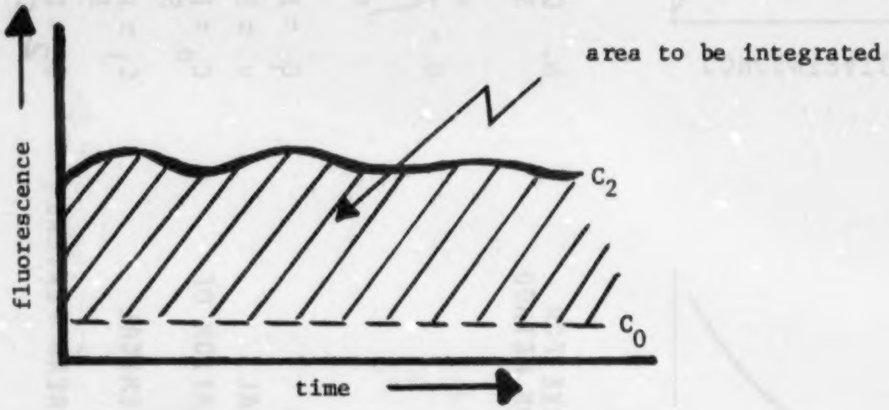
C_2 = IS INSTANTANEOUS STREAM CONCENTRATION

FIGURE 44

CONSTANT RATE AND SLUG INJECTION METHODS



For slug injection, the bottom value $\int (C_2 - C_o)dt$ can be determined graphically by plotting fluorescence over time and integrating between C_o and C_2 manually with a planimeter:



The advantage of slug injection is sophisticated equipment is not needed for injection. The disadvantages of slug injection are:

- It is only good in steady flows,
- The entire pulse must be sampled.

The error rate of either of these methods is ± 3%. For more information, see reference 9, 10, 27, and 28.

EXOTIC METHODS

There are three rather exotic methods for measuring flow. They use the principles of electromagnetic, acoustic, and electrical energy. Usually, they will neither be seen nor used by the NPDES inspector, but their presence here completes this manual.

Electromagnetic Flowmeter

An electromagnetic flowmeter, like the one shown in Figure 45, works on the principle of Faraday's Law, which says that the voltage is directly proportional to the velocity and to the magnetic field strength. This is measured by electrodes in contact with the water or wastewater. The conductor must be perpendicular to the electrical field, induced by an electromagnet.

This method is good in pipes with diameters from 2 to 24 inches (0.167 to 2 ft). Its error is ± 1%, and it has no moving parts. A severe disadvantage is the buildup of solids on the electrodes can cause error. Thus frequent cleaning of the flowmeter is essential. Vacuum or air entrainment may also cause error.

Acoustic Flowmeters

Acoustic flowmeters operate on the principle that the difference in time of arrival of two simultaneously created acoustic or sound pulses traveling in opposite directions through the water can be related to the velocity of flow.[9]

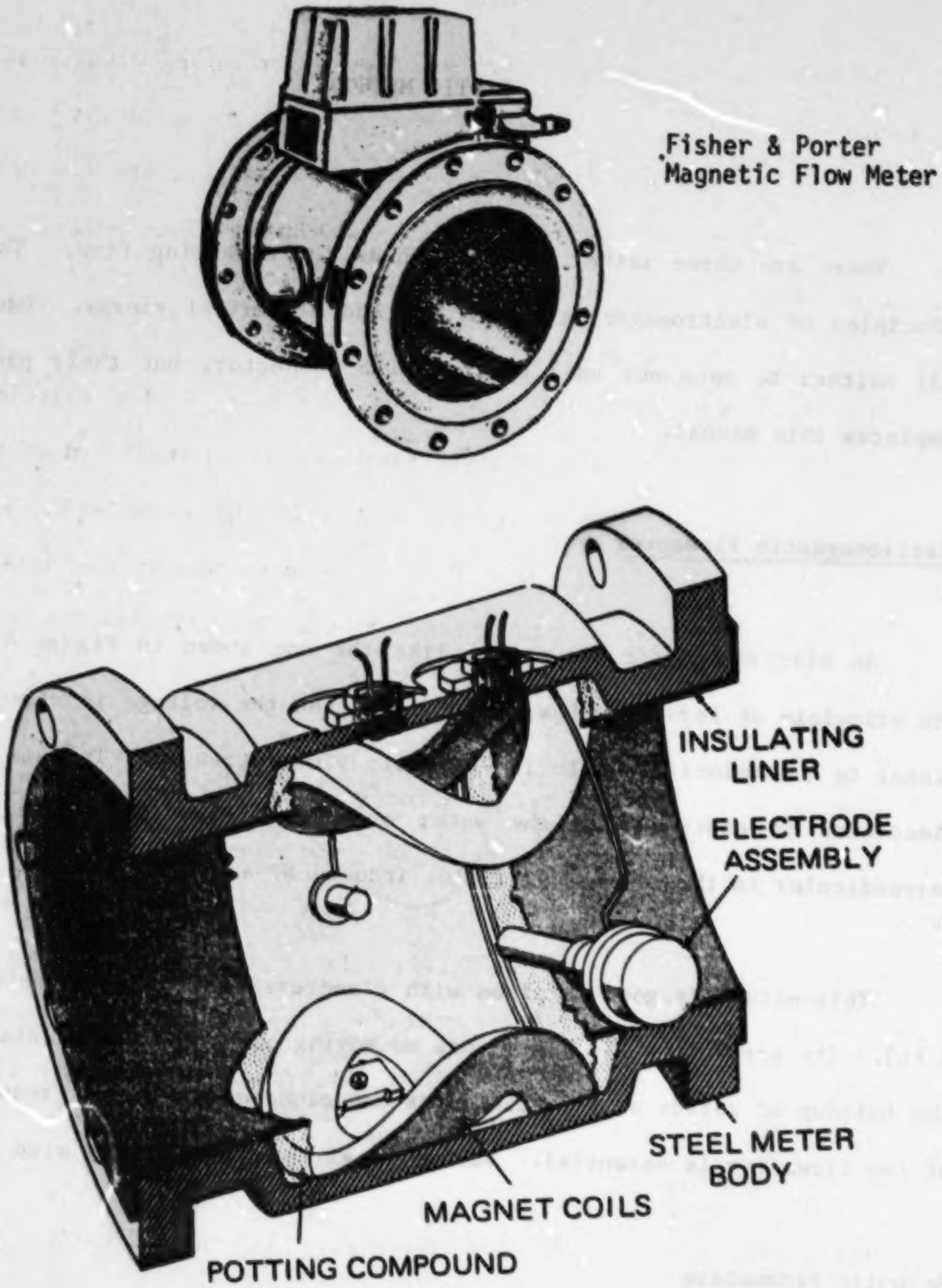

Fisher & Porter
Magnetic Flow Meter
INSULATING LINER
ELECTRODE ASSEMBLY
STEEL METER BODY
MAGNET COILS
POTTING COMPOUND

FIGURE 45

TYPICAL MAGNETIC FLOW METER

They also make use of the fact that sound waves traveling downstream propagate at higher velocities than those traveling upstream.* A mathematical solution and picture of an acoustic flowmeter is Figure 46. There are two generic types of acoustic or ultrasonic meters available--a Doppler-effect meter, and a "Delta traveling-time meter"(Author's terminology).[27]

The first type of sonic flow meters operates on the principle of the Doppler effect. A sonic wave of given frequency is emitted through the flowing stream. The effect of the moving wastewater on the sound wave results in a shift of the wave frequency which in turn is detected by the receiver. The degree of frequency shift is proportional to the flow velocity. A conversion to flow velocity is done electronically.

The second type of sonic flow meter works on the principle of the difference in traveling times for two identical sound waves which travel at different angles. One wave travels along the flow direction, while the other wave travels against the flow direction. The differential traveling time reveals the flow velocity. Therefore, this type of sonic meter generally has the sonic emitter and receiver aligned at an angle which is oblique to the direction of flow. (See Figure 46).

Ultrasonic flow meters are increasing in their application in flow measurement in POTWs for both wastewaters and sludges. Sonic flow meters have been successfully coupled with Venturi tubes for accurate flow measurements.

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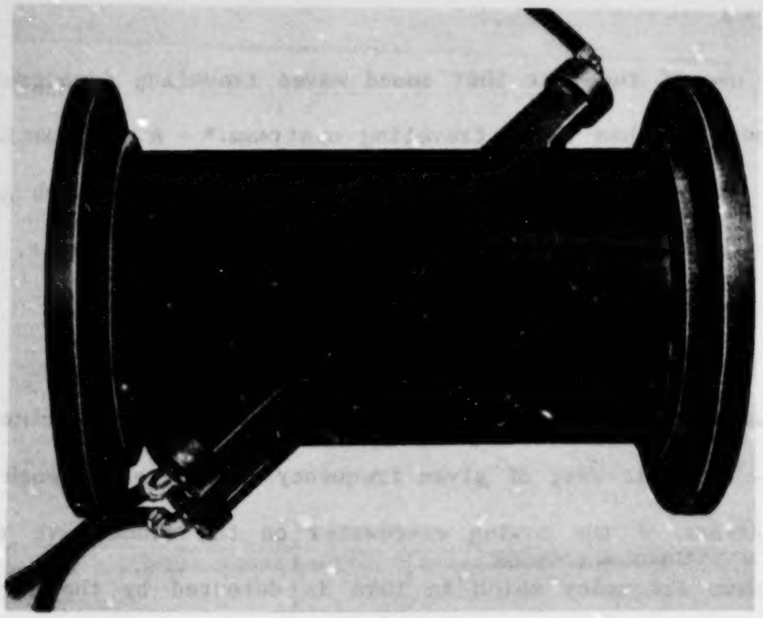
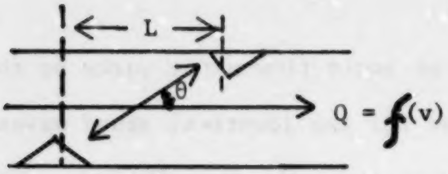
* BIF Company



ULTRASONIC FLOWMETER



$$\Delta T = t_u - t_d, \quad t_{avg} = \frac{t_u + t_d}{2}$$

$$V \approx \frac{\Delta T}{t_{avg}} - \frac{L}{2\cos\theta}$$

$$t_u = \frac{L}{C - V\cos\theta}, \quad t_d = \frac{L}{C + V\cos\theta}$$

t= traveltime, C = velocity of sound (≃ 1088 fps), V = flow velocity, L = pathlength. The t_u and t_d are measured along the diagonal acoustic path. Sound waves traveling downstream propagate at higher velocities than those traveling upstream.

FIGURE 46

Ultrasonic flow meters do not come in direct contact with the flow, therefore, are less susceptible to the foulings. Sonic flow meters are sensitive to and compatible with a wide range of flow rates.

Vacuum conditions (which result in gas production) or an entrainment in the pipes may produce false readings by the sonic flow meter. Sonic flow meters themselves may be affected by environmental conditions such as high temperature, rapid temperature fluctuations, or humidity.

Electrical Methods

There are three electrical methods for measuring wastewater flow. A thorough discussion is beyond the scope of this manual, especially since the NPDES inspector is unlikely to encounter these in the field. All of their performances in accurately measuring wastewater flow are hampered by high suspended solids loads. The methods are:[27]

- Conductivity cells
- Hot-wire anemometers
- Warm-film anemometers

SUMMARY

This manual is intended to serve as a guide for the NPDES inspector. Often he/she will encounter alien flow situations where he/she will be faced with difficult problems in calculating wastewater flows. As many practical ideas as possible are incorporated to aid the inspector in these situations. Example problems are included to highlight important concepts in practicality.

However the manual does not include every detail related to flow measurement. The references should serve to augment the material presented here if more information is required by the NPDES inspector.

A final word about safety is appropriate at this point. Many field situations are extremely dangerous, especially concerning:[19]

- Traffic diversion problems,
- Hazardous and toxic materials,
- Sewer gas buildup and the risk of explosion,
- Theft of equipment kept on-site,
- Deep manholes as safety hazards,
- Noise in confined areas.

In these flow measurement methods, environmental in situ factors not only have a tendency to shorten equipment life, reduce measurement accuracy, and lengthen necessary field time, but they also could potentially jeopardize human life.

Be careful. Observe standard safety procedures at all times. And above all, use your common sense!

FOOTNOTES

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FOOTNOTES
(continued)

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APPENDIX

I

PUMPING STATIONS AS FLOWMETERS[5]*

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In the normal operation of wastewater handling facilities, and especially when making infiltration/inflow analyses, it is desirable and even necessary to measure the amount of wastewater handled by a pumping station. Ten State Standards[1] lists flow measurement capability as a requirement for new pumping stations. State agencies reviewing plans for pumping stations generally interpret this requirement as applying only to the larger stations, and many older or smaller pumping stations are without any means for measurement.

The author has developed a method by which the flow of any electrically driven pumping station may be measured with accuracy that will satisfy most requirements. The method is based on the proposition that the mass volume of water (wastewater) pumped by any given station is directly proportional to the consumption of electrical energy in kilowatthours. This means that, for the duration of time under consideration, for example, 1 hr. or 1 day, the total through-put of the station in gallons is proportional to the kilowatthours consumed during that period. The average flow during the period under consideration may be determined by dividing the total gallons by the time period. Thus, the kilowatthour meter may be read hourly, daily, or weekly and the average flow computed for the time interval desired.

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[1] "Recommended Standards for Sewage Works." Great Lakes-Upper Mississippi River Board of State Sanitary Engineers.

*Corrections were made by this author.

There remains, of course, the matter of determining the constant of proportionality for a particular station in terms of gallons per kilowatthour. This constant must be determined experimentally for each pumping station because each is unique in terms of static head, friction head, pump efficiencies, motor efficiencies, and station auxiliaries. This paper is devoted largely to an explanation of the method of determination of station constants. It will show how a few simple experimental observations may be made on a pumping station and how, from these, the required station constant may be derived.

For purposes of this explanation, a conventional dry pit pumping station may be visualized as having two identical electric pumps drawing from a separate wet well. The pumps are assumed to be equipped with controls that permit one pump to pump the wet well down and then shut off. On filling up a second time, the other pump performs the pumping down; the two pumps alternate unless the water level in the wet well reaches the alarm level and calls for duty from both pumps. It is also assumed that the station is fitted with other powerconsuming auxiliaries such as sump pump, dehumidifier, and electric lights.

The influent sewer runs steadily until the water level in the wet well reached the "pump-on" level. Then the duty pump starts and the water is pumped down until the water level reaches "pump-off." This is called one pump cycle. During one pump cycle it is necessary for the pump to pump out the volume stored in the wet well between pump-on and pump-off plus the flow of the influent sewer. This may be expressed by an equation:

$$V_p = V_s + Q_i t_p \qquad (1)$$

in which:

V_p = volume pumped per cycle (gal),

V_s = storage volume of the wet well,

Q_i = flow of the influent sewer (gal/sec), and

t_p = time that the pump runs per cycle (sec).

V_s, the storage volume, may be easily determined by measuring the difference between the pump-on and pump-off levels (drawdown) and multiplying by the cross-sectional area of the wet well.

Thus, for a circular wet well,

$$V_s = 0.7854 \times D^2 \times d \times 7.48 \qquad (2)$$

in which:

V_s = storage volume (gal),

D = diameter of wet well (ft),

d = the drawdown (ft)

0.7854 = the constant, $\pi/4 \left[\text{from } A_{\text{circle}} (\pi)(D^2/4)\right]$

7.48 = a constant (gal/cu ft).

For a rectangular wet well, the equation would become,

$$V_s = l \times w \times d \times 7.48 \qquad (3)$$

in which:

l = length of the wet well (ft),

w = width of the wet well (ft)

d = drawdown (ft), and

7.48 = a constant (gal/cu ft).

Knowledge of the storage volume allows one to measure the flow of the influent sewer. This is done by simply observing the time required to fill the wet well. As soon as the duty pump shuts off, the stop watch is started and the wet well starts to refill. When the wet well is full, the next pump will start and the fill time is recorded.

To calculate the flow of the influent sewer the following equation is used:

$$Q_i = \frac{V_s}{t_f} \tag{4}$$

in which:

Q_i = flow of the influent sewer (gal/sec),

V_s = storage (gal), and

t_f = fill time (sec).

Equation 1 still requires one further parameter, t_p the pumping time for one pump cycle. This is the time that one pump requires to pump the wet well down one time. This time is observed with a stopwatch and recorded.

Now all of the data necessary to compute V_p, the volume pumped per cycle, are available.

The use of electrical energy also should be considered. An electric watthour meter operates only when power is flowing through it. Ordinarily, it indicates the amount of electrical energy consumed by a series of dials that register the cumulative kilowatthours consumed. There is also an aluminum disk that rotates when power is being used. There may also be a demand scale and pointer on some meters, but this analysis is not concerned with demand.

The amount of electrical energy through a kilowatthour meter is directly proportional to the total number of rotations of the aluminum disk. The constant of proportionality is ordinarily shown on the face of the meter and is designated K_h. The dimensions of this constant are watthours per revolution. For example, if a meter has a K_h of 14.4, 14.4 watthours of electricity are consumed during on revolution. It would therefore take 1,000/14.4 = 69.44 revolutions to indicate usage of 1 kwh. The ability to measure the use of small amounts of electrical energy by observing the meter disk makes it possible to determine the amount of energy used by the lift station for one pump cycle. It is simply necessary to observe the number of revolutions made by the meter disk during one pump cycle.

The amount of energy used for one pump cycle is as follows:

$$E_1 = \frac{n \times K_h}{1,000} \qquad (5)$$

in which:

E_1 = energy used per pump cycle (kilowatthours),

n = number of revolutions made by meter disk during one pump cycle,

and

K_h = meter constant (watt-hours per revolution).

The factor of interest here is the station constant expressed in gallons per kilowatthour. When this is known, it is possible to determine flow for any period for the lift station simply by reading the meter dials on two separate occasions, subtracting the readings to get the amount of electrical energy used, and then multiplying this figure by the station constant to get total gallons through the station. By dividing this result by the time interval between the two meter readings, the average flow during that interval is obtained.

The foregoing may be reduced to a single equation:

$$V_p = V_s + Q_i t_p \tag{6}$$

(repeat Equation 1)

$$V_p = V_s + \left(\frac{V_s}{t_f}\right) t_p \tag{7}$$

(substituting for Q_i, Equation 4)

$$C = \frac{V_p}{E_1} = \frac{V_s + \dfrac{V_s t_p}{t_f}}{\dfrac{n \times K_h}{1{,}000}} \tag{8}$$

(dividing by Equation 5)

$$C = \frac{1{,}000\ V_s \left(1 + \dfrac{t_p}{t_f}\right)}{nK_h}$$

(simplifying) (9)

in which:

C = station constant (gal/kwh),

V_s = wet well storage volume (gal),

t_p = time required to pump down one time (sec),

t_f = time required for wet well to fill (sec),

n = number of revolutions of meter disk per pump cycle, and

K_h = meter constant (watthours per revolution)

The above discussion describes the method by which the lift station is calibrated for one pump. A similar set of observations should be made on the other pump in the station, and, ordinarily, two identical pumps will exhibit similar power requirements. A large difference is a warning of trouble somewhere in the system.

Because the pumps operate alternately, a true averaging takes place and the two values of the station constant may be averaged to represent the station under actual alternating pump service. This condition is true for lightly loaded stations or for dry weather conditions. In wet weather or for overloaded stations, both pumps may run together. A new station constant must be determined for this condition. It is done in the same manner as described, except that both pumps are turned on while the meter readings are taken. As might be expected, when both pumps are manifolded into a common force line, the station constant will be lower than with single pumps; that is, there will be fewer gallons pumped per kilowatthour of electrical energy used. When both pumps operate together, the force line tends to choke and the total dynamic head loss is greater, leading to poorer pumping efficiency. When making flow measurements, the researcher will use either the average constant of two pumps running separately or the constant determined when both pumps run together, whichever is appropriate to the actual pumping situation.

Once the constants for the station have been determined, it is reasonable to assume that they will remain unchanged unless the station is modified in some way. To determine flow it is necessary to read only the kilowatthour dials (not the disk) at the beginning and the end of the period desired. The power company will cooperate in showing the operator how to read these dials. It should be informed of the purpose of the readings. Sometimes, the meter enclosures will be locked or sealed against vandalism and it may be necessary to ask the power company for access to the meter. The average flow for the period under observation many be found from the expression

$$Q_{avg} = \frac{C(E_{t1} - E_{to})}{t} \qquad (10)$$

in which:

Q_{avg} = average flow for the period,
C = station constant (gal/kwh),
E_{to} = kilowatthour reading at the beginning of the period,
E_{t1} = kilowatthour reading at the end of the period, and
t = elapsed time between observations E_{to} and E_{t1}

The units of Q_{avg} will depend on the units chosen for t. If t is in minutes, Q_{avg} will be in gallons per minute; if t is in days, Q_{avg} will be in gallons per day. Ordinarily, it will not be practicable to read the meter at an interval shorter than 1 hour because the power consumed will be too small to produce a reliable reading. If an instantaneous flow is desired, the reader should refer to Equation 4 and compute the flow of the influent sewer, Q_i.

For purposes of measurements for infiltration/inflow analyses, it is convenient to read the electric meter once a day, recording the time of day. From each pair of readings, the daily flow may be computed and a trend identified from dry and wet weather readings.

Attention should be paid to any meter multipliers. These multipliers are devices attached to the meters by the power company to extend their range and are often tagged on the face of the meter. If a meter has a multiplier, the multiplier value must be added to both the K_h and the kilowatthour readings.

The pumping station auxiliaries use about 5 percent of the total energy consumed by the station, and this component is fairly constant and independent of the pumping rate. Because the station is calibrated in terms of gross energy consumed, some error will be introduced by not significantly so. For practical purposes, it may be neglected.

The method is best illustrated by an example. The following parameters may be assumed for a lift station such as the one described above:

Diameter of wet well = 6.0 ft

Drawdown in wet well = 2.2 ft

Time for wet well to fill = 318 sec

Time for wet will to empty = 99 sec (time pump runs/cycle)

Electric meter constant K_h = 14.4 watthours/revolution

Electric meter disk revolutions for one pump to pump down wet well = 15.34 revolutions

$$V_s = 0.7854 \times D^2 \times d \times 7.48$$

$$V_s = 0.7854 \times (6)^2 \times 2.2 \times 7.48$$

V_s = 465 gal, wet well storage

$$C = \frac{1{,}000\ V_s \left(1 + \frac{t_p}{t_f}\right)}{nK_h}$$

$$C = \frac{1{,}000 \times 465 \left(1 + \frac{99}{318}\right)}{15.34 \times 14.4}$$

C = 2,760 gal/kwh, station constant

The above calculation determines the station constant. The station is now observed on 2 successive days and the following readings taken from the electric meter dials:

February 25, 1975 11:10 a.m.

67532.0

February 26, 1975, 8:47 a.m.

67553.3

67553.3 - 67532.0 = 21.3 kwh consumed

Time interval = 1,297 min.

$$\frac{1{,}297 \text{ min}}{1{,}440 \text{ min/day}} = 0.9007 \text{ days}$$

$$Q_{avg} = \frac{C(E_{t_1} - E_{t_o})}{t}$$

$$Q_{avg} = \frac{2{,}760\ (67553.3 - 67532.0)}{0.9007}$$

$$Q_{avg} = 65{,}269 \text{ gal/day}$$

Once the technique is learned, it is convenient to prepare a program for computer or programmable calculator to assist in reduction of data. The reduces labor and mistakes in calculation and makes the process almost routine so that the engineer may focus his full attention on the meaning of the results thus obtained.

Pneumatic ejector stations may also be calibrated by a slightly different technique. Most pneumatic ejectors used in municipal wastewater service are of the duplex type, consisting of two pressure vessels, one or more air compressors, and, sometimes, an air storage tank. Essentially, these stations work by admitting wastewater into one of the pressure vessels and, when it is full, applying air pressure to exhaust the water. Each time a "pot" discharges, it discharges a known amount of water, the volume of one pot. The researcher may determine this volume from the geometry of the pot. To calibrate such a station, it is simply necessary to relate the amount of electric energy used to the amount of water discharged. The following example illustrates this readily. An ejector station has two 75-gal pots. It was observed for a 30-min period, and, during this time, each pot discharged 16 times. The electric meter disk made 120.85 revolutions during this period, and the meter had K_h of 10.8.

16 + 16 = 32 pot discharges in 30-min test period.

V_p = 32 pots x 75 gal/pot = 2,400 gal discharged in 30-min test

$$E_1 = 120.85 \text{ revolutions} \times 10.8 \, \frac{\text{watthours}}{\text{revolution}} \times \frac{1 \text{ kwh}}{1{,}000 \text{ watthours}}$$

= 1.305 kwh consumed during 30-min test period

$$C = V_p/E_1$$

$$C = \frac{2{,}400 \text{ gal}}{1.305 \text{ kwh}} = \begin{array}{c}1{,}839 \text{ gal/kwh} \\ \text{station constant}\end{array}$$

Ejector stations must be carefully observed to make sure that the pots fill completely each cycle and do not fire prematurely. Pots that are triggered by electrode generally will not fire until the liquid level reaches the upper electrode. Some ejectors are equipped with timers, and error will be introduced if the pots do not fill completely.

As with any experimental technique, this method needs to be applied judiciously. When a station is overhauled or repaired or the controls adjusted, the researcher should recalibrate. Used with discretion, the method provides an inexpensive way to monitor flows and makes the power companies' records a source of data for determiring historical growth of the flows in the system.

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